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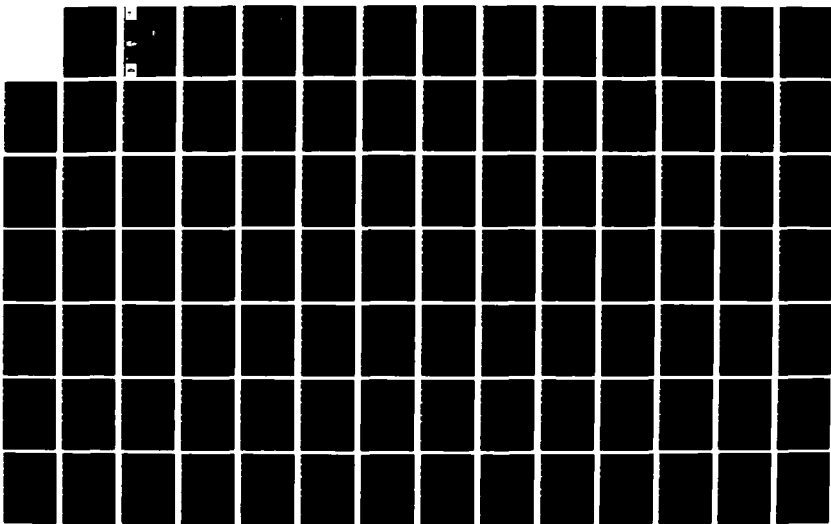
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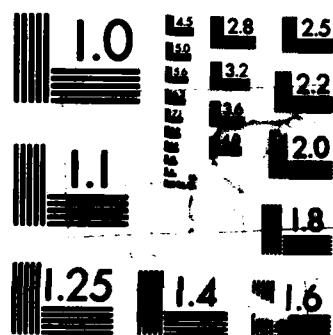
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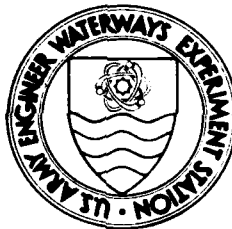
by

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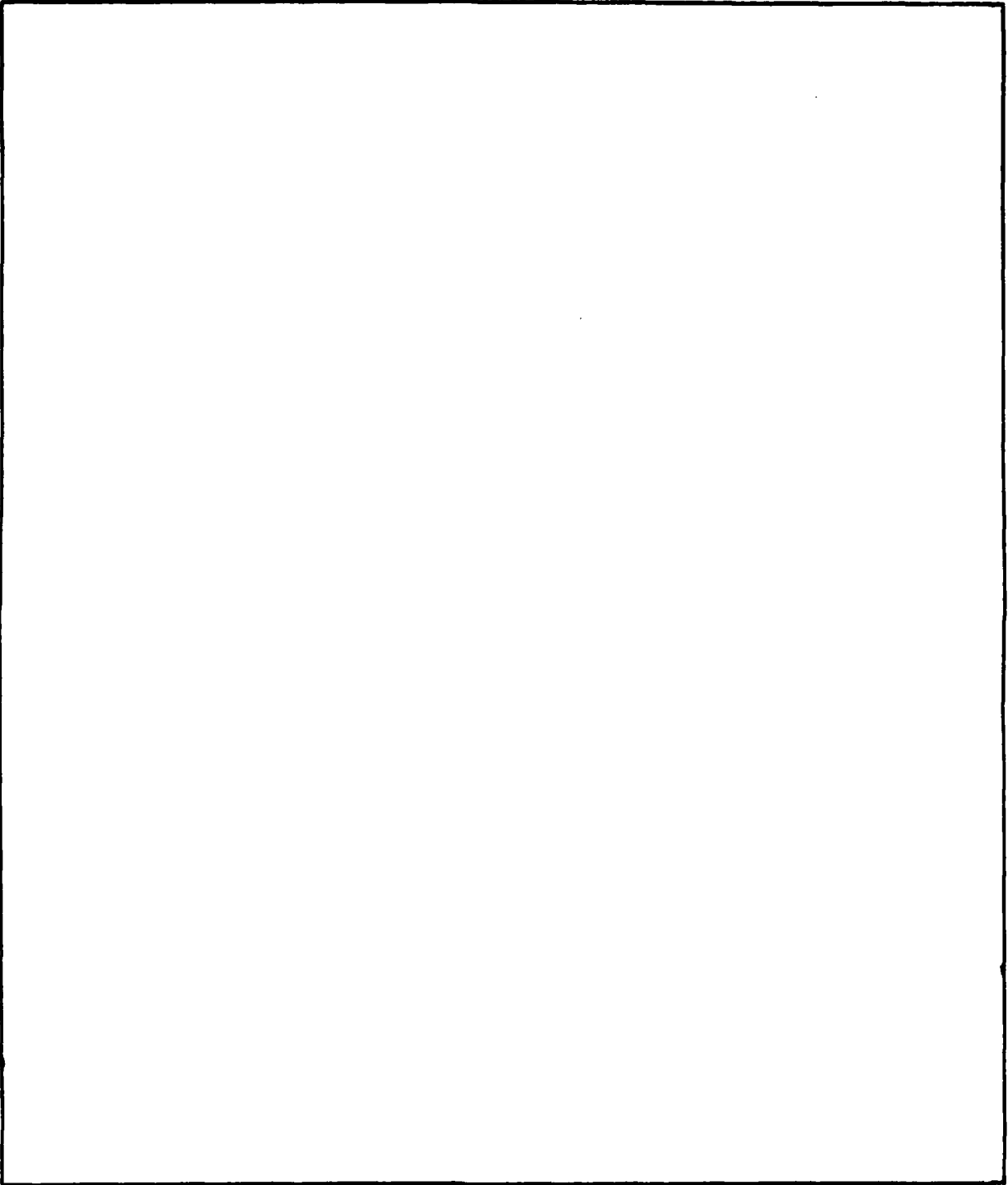
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PREFACE

This report was prepared by Gregory B. Baecher of NEXUS Associates, Wayland, Massachusetts, with assistance from D. DeGroot, C. Erikson, under Contract DACW39-83-M-0067. It was part of work done by the US Army Engineer Waterways Experiment Station (WES) in the Civil Works Investigation Study sponsored by the Office, Chief of Engineers, US Army. This study was carried out under Work Unit No. Civis 32221, entitled "Probabilistic Methods in Soil Mechanics," during the period October 1983 to September 1985. Mr. Richard Davidson was the OCE Technical Monitor.

This document reports findings of an investigation into the use of various statistical methods for geotechnical engineering aspects of new dam projects. The intent of the investigation was to identify specific statistical methodology which could be beneficially adopted by the US Army Corps of Engineers (USACE) on future projects. The benefit provided by these methods might include increased productivity in engineering, increased efficiency in dam design, improvements to construction quality assurance, or increased project safety.

As a vehicle for investigating statistical methods, the study used an existing USACE project as a test case. Data from that project provided the substantive material upon which statistical methodology was exercised. It was not the intent of the present work to reanalyze the test case project, nor did it do so. Nothing in this report should be construed as a review of the test case project. The aspects of the test case project selected for statistical analyses were those for which it appeared that statistical methodology might provide new and better ways of doing geotechnical engineering. These are not necessarily the critical aspects of dam design, nor are they necessarily those associated with dam safety.



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publication of this report. Dr. Robert W. Whalin was Technical Director.

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UNCERTAINTY ANALYSIS FOR DAM PROJECTS

PART I: INTRODUCTION AND OBJECTIVES

This report evaluates the application of statistical methods to geotechnical engineering aspects of new dam projects. This is the final summary report of work under Contract DACW39-83-0067, "Uncertainty analysis for dam projects". Work under contract DACW39-83-0067 began October 1, 1983 and was completed on September 30, 1985. In total, approximately two (2) man-years effort was allocated to the project.

The goal of the work was to assess potential applications of statistical methods to new dam projects of the Corps of Engineers. To do this, two main objectives were identified:

- First, to better understand the sources of uncertainty in dam engineering, and to identify phases within the course of a project where statistical methods would provide benefit over present methods; and
- Second, to tailor or develop practical statistical procedures for dealing with geotechnical data in dam engineering, and to compare these methods with current USAE procedures.

Under the second objective, those methods which appear to have promise have been summarized in an instructional form and published as separate reports.

The coverage of this report is limited to statistical methods, as for example, as they are applied to data analysis, error propagation in engineering models, quality assurance, and other aspects of geotechnical engineering. The report does not cover probabilistic design or probabilistic risk assessment.

Three additional reports accompany this final report. These are,

1. Statistical Data Analysis Geotechnical Data (Contract Report GL-87-).
2. Error Analysis for Geotechnical Engineering (Contract Report GL-87-).
3. Statistical Quality Control Engineered Embankments (Contract Report GL-87-).

Detailed development of the statistical methods underlying geotechnical data analysis, error propagation, and quality control and assurance is deferred to the manuals.

Approach

These objectives were approached through the study of a specific, completed Corps project. Carters Dam, Georgia, was chosen by the Corps to serve as this case study. The Carters project is under the jurisdiction of the Mobile Engineer District, South Atlantic Division, which has provided access to data files and guidance by personnel familiar with the project. Carters Dam was chosen as a typical USAE project. It was not chosen to be peculiarly appropriate to statistical methods.

Background

Considerable attention in recent years has been focused by the research community on the development of statistical methods for geotechnical engineering. These methods are intended to rationalize the treatment of uncertainty in dam projects, and great promises have been made by their promoters. Yet, statistical methods have not been widely applied to geotechnical aspects of dam projects, and no systematic evaluation of their practicality and benefit has been undertaken. The present project was

undertaken to provide this systematic evaluation, and to make recommendations on what the USAE could do to make effective use of the new methods.

The introduction of statistical methods into geotechnical practice in dam engineering is motivated by two principal factors:

¶ First, increasing amounts and detail of site characterization data, increasing sophistication of engineering analysis, and increasing computerization are making traditional approaches to data analysis obsolete. Statistical data analysis offers the prospect of increasing the efficiency of data manipulation while simultaneously improving the power of inferences drawn from data.

Traditionally, geotechnical engineering has used an informal approach to data collection, analysis, and manipulation. Total numbers of data on any project have been small. As a result, visual inspection of data and intuitive estimation of engineering parameters have been both economically feasible and technically justified.

Today, geotechnical engineering is facing a number of challenges. Important among these are (a) computerization, and (b) new in situ testing technology. Computerization has led to vast improvements in the ability to mathematically model physical processes. However, finite element techniques, dynamic analyses, complex constitutive relations and other new modeling tools are data-hungry. They have led to rapid escalation in the need for geotechnical data. At the same time, improvements in in situ technology, including continuous profiling devices such as the piezo-cone penetrometer, have led to order-of-magnitude increases in the numbers of data developed by site characterization programs. These data are too voluminous to analyze by inspection. Computerized automated data processing (ADP), however, provides a

vehicle both for managing these large data sets and providing the analysis necessary to generate input parameters for advanced models.

¶ Second, risk analysis of civil works has become increasingly important as regulatory reviews have become stricter, and as public concern for the consequences of engineering failures has become widespread.

The demand for quantitative risk analysis of dam safety is now driven by forces outside the geotechnical engineering community. To a large extent the successful implementation of probabilistic risk assessment (PRA) in regulatory processes involving nuclear plant safety, environmental impacts, and other areas of national concern has led to momentum for applying PRA techniques to other areas of public safety.

PRA methodology, however, has developed primarily in the aerospace, nuclear, and manufacturing industries. As a result, much of the methodology is inappropriate to the needs of geotechnical engineering practice. Basic developments in data analysis, reliability modeling, quality control, and related technology will have to be made before PRA can be routinely applied to geotechnical engineering aspects of new dam projects

Each of these factors--computerization and increased numbers of data, and demands for probabilistic risk assessment--requires a better understanding of the use of statistical methods for describing and analyzing geotechnical engineering data than the profession now enjoys. Specific to geotechnical engineering aspects of new dam projects, three issues need to be addressed:

1. Can geotechnical data on new dam projects be described and analyzed statistically (i.e., are geotechnical data mathematically well-behaved)?

2. Is currently available statistical technology suitable for the geotechnical data analysis problems faced on new dam projects?
3. How might the introduction of statistical technology to USAE design practice for geotechnical engineering aspects of new dam projects affect efficiency and safety?

Resolving these issues does not involve an analysis of computerization or risk assessment per se, but rather consideration of the more limited question of the applicability of statistical methods to geotechnical data. Answering the statistical question has direct benefits of its own, but also intends to serve as a technological basis for addressing the role of risk assessment.

Objectives

Task level objectives of the work have addressed individual uses to which statistical methods may prove beneficial to Corps projects. These objectives comprised eight (8) tasks (Figure 1). The first three involved data collection and evaluation for the study project, and identification of points within the life of the study project where statistical methods might have provided insight or efficiency beyond present practice. Tasks 4 through 7 developed and applied specific statistical procedures to the study project and evaluated costs and benefits of the methods relative to the procedures that were used at the time. The final task identified and recommends those statistical procedures judged beneficial.

TASK 1 --Collection of project data

The Carters project was reviewed to create an inventory of data, the point in the project life at which data were collected, the respective

purposes of collecting the data, and the project decisions resting on the data.

TASK 2 --Description of sources and components of uncertainty

TASK 2 analyzed the assumptions and decisions in the case study that either, (1) had a major influence on engineering uncertainty, or (2) were significantly influenced by engineering uncertainty during the project. Special consideration has been given to decisions that were made about or that resulted from data collection. Work under Task 2 lead to a classification of types of engineering uncertainties in dam projects, and to an understanding of the influence on these uncertainties of data collection and analysis.

TASK 3 --Survey and summary of statistical approaches

Task 3 work surveyed statistical procedures and methods proposed for use in geotechnical data analysis (i.e., in the geotechnical literature) or deemed applicable. This work included a review of previous applications to geotechnical problems, with emphasis on dam engineering.

TASK 4 --Evaluation of Statistical Methods in Application to Project Conception, Planning and Exploration Layout

TASK 5 --Evaluation of Statistical Methods in Application to Site Characterization

TASK 6 --Evaluation of Statistical Methods in Application to Design

TASK 7 --Evaluation of Statistical Methods in Application to Construction and Operation

These four tasks involved tailoring and developing statistical methods to fit the needs of the Carters project.

TASK 8 --Prepare Recommendations and Instructional Reports

Work under Task 8 summarized the study and recommendations on the use of statistical methods in USAE dam projects. Three separate reports were prepared for statistical methods recommended by the study.

Organization of This Report

In the organization of this report, Part II surveys the results of work on Task 3. Part III describes the project data that were reviewed under Task 1. Part IV summarizes the results of Task 2 work and many of the results of Tasks 4 through 7 work. Part V summarizes the results of Tasks 4 through 7 in how statistical methods, retrospectively, might have aided engineering for the Carters Project. Parts VI and VII report the conclusions and recommendations of Task 8.

Task
<u>Year 1</u>
<ol style="list-style-type: none"> 1. Collect project data 2. Identify sources and components of uncertainty 3. Review and summary of statistical approaches 4. Evaluation of statistical methods for project conception, planning and exploration layout
<u>Year 2</u>
<ol style="list-style-type: none"> 5. Evaluation of statistical methods for site characterization 6. Evaluation of statistical methods for design 7. Evaluation of statistical methods for construction and operation phases 8. Recommendations

Figure 1. Task Objectives.

PART II: STATISTICAL METHODS IN GEOTECHNICAL ENGINEERING

Part II provides an overview of the use of statistical, probabilistic, and risk analysis methods in geotechnical engineering, with particular attention to new dam projects. Statistical methods are highlighted. Current directions in research and development are reviewed at the end of Part II.

Statistics, Probability, and Risk

Probabilistic and statistical methods in geotechnical engineering are of several distinct forms, having different purposes. As a matter of convenience, they may be divided into four groups by the methods they use and the questions they answer:

- Probabilistic techniques,
- Statistical methods,
- Risk assessment, and
- Economic optimization (decision analysis).

Probabilistic Techniques & Reliability

Probability theory is a branch of mathematics which can be used to characterize uncertainties about engineering parameters and to describe the relations among such uncertainties. In this way probability theory is similar to engineering mechanics. The theory is internally consistent, and once the characteristics of a set of random variables are defined all further results of probabilistic modeling--as in engineering mechanics--follow necessarily.

Probability theory is used in geotechnical engineering essentially to propagate uncertainties about engineering parameters or variables through geomechanical models to draw conclusions on uncertainties in

the predictions of those models. For example, given information about the uncertainty of soil conditions, probability theory can be used to calculate the uncertainty of bearing capacity predictions made by Terzaghi's superposition formula or settlement predictions made by 1D consolidation theory.

Geotechnical reliability analysis is the application of probabilistic models to geotechnical systems in order to replace conventional safety indices, for example the factor of safety F , with indices based on probabilistic descriptions. The most common probabilistic index is the so-called "probability of failure," p_f . Usually, this is defined as the area under the probability distribution function of predicted performance overlying those values of the predicted performance which are defined as "failure." Contrary to appearance, this index is not in fact a prediction of the rate at which facilities perform adversely, but has rather to do with uncertainty in calculations. That is, p_f is the probability that errors in the selection of parameter values for input to the engineering calculations might be so large or in such combination that the analysis should be yielding a prediction of adverse performance. Other common probabilistic indices are based on moments of the distributions of predicted performance. The most important of these are the first-order second-moment reliability index β_{FOSM} (Cornell, 1971), the Hasofer-Lind index β_{HL} (Hasofer and Lind, 1974), and the second-moment reliability index β_{SM} (Rackwitz, 1981).

In many cases, the purpose of reliability analysis is to predict rates of actual failure and not simply safety indices. In this review, such cases are considered as risk assessment.

Statistical Methods

Statistical methods are a set of techniques, sometimes ad hoc, for drawing inferences from observations. These methods use probability theory as a means for describing variability and in some cases uncertainty, but they are not themselves axiomatically based. That is, statistics is not a formal branch of mathematical theory.

Statistical methods are used in geotechnical engineering primarily to analyze data on site conditions and environmental loads. To some extent they are applied to validating model predictions against observed performance. This latter use might be expected to increase as probabilistic analysis become more widespread. The intent of statistical analysis in geotechnical applications is to make efficient use of data and to provide the probabilistic characterization of uncertainty necessary for reliability modeling or risk analysis. Increasingly, statistical methods are also being used to plan efficient "scientific" experiments for gathering information, validating models, or controlling construction quality.

Two primary approaches to the use of statistical methods in geotechnical engineering derive from two separate schools of thought on the meaning of uncertainty. One school of thought holds that uncertainty reflects the relative frequency with which certain events occur or with which, say, soil properties are distributed. This approach is called frequentist. The other school of thought holds that uncertainty reflects subjective belief or credibility about natural phenomena, as for example, in considering the likelihood of one-time events such as faulting at a particular site. This approach is called Bayesian.

The distinction between frequentist and Bayesian statistics is important both philosophically and practically. Frequentist theory defines probability

as the frequency of occurrence in series of similar "trials," and thus uses probability to describe variability. Bayesian theory defines probability as belief or credibility, and thus used probability to describe mental uncertainty. Inference procedures differ between the two approaches, and each has an appropriate role to play in dealing with geotechnical engineering problems.

Risk Analysis

Risk analysis in its meaning here is the effort to bring a characterization of all relevant uncertainties together in an analysis to assess the aggregate uncertainty facing a "decision maker." This uncertainty is of many types. One part of a risk analysis is forecasting rates at which real facilities fail. Another is assessing systematic errors which manifest as correlated failures. A proper risk assessment leads to predictions of rates of failure and a quantification of the uncertainty in those predicted rates.

Risk assessment is typically a eclectic mixture of statistical analysis, probabilistic modeling, expert opinion, and pragmatism. Its use to date in geotechnical engineering has been limited and often proprietary, for example, in evaluating risks for insurance underwriting. An increasing area of use is in regulatory licensing and evaluation of siting hazards for power plants and hazardous facilities. It appears likely that risk analysis will also become more widespread in the design of dams and other civil projects.

Decision Making and Optimization

Optimization of design or project decisions by balancing risk against cost requires not only risk assessment but also an analysis of the costs accruing to failures or other adverse performance. In many cases such failure costs

involve only economic attributes, but in others they involve costs which are noncommensurate with monetary attributes such as life loss, environmental degradation, and social disruption. Decision analysis and optimization attempt to quantify the consequences of facility failures, combine these quantifications with assessments of their associated probabilities, and identify design or project options that are in some sense optimal.

In geotechnical engineering decision analysis approaches have been often discussed but seldom implemented in a serious and comprehensive way. Applications have tended to emphasize either careful assessment of consequences or careful assessment of probabilities, but rarely both. The better of the applications of decision analysis in geotechnical engineering for the most part have dealt with regulatory problems such as power plant siting, in which the principal uncertainties of concern do not deal with soil or rock mechanics problems.

Uncertainty in Geotechnical Engineering

Uncertainties in geotechnical predictions arise from several sources. Some of these can be enumerated and quantified, some cannot. For any engineering analysis, formulating the problem involves hypotheses and decisions upon whether those hypotheses are credible. Such tasks are not amenable to mathematical logic. Thus uncertainties associated with them can only be treated subjectively.

In an approximate way the uncertainties entering engineering analyses are of five types:

1. Site conditions and parameter estimation errors,
2. Loads
3. Model inaccuracies
4. Construction and quality control, and
5. Omissions and gross errors.

Not all these can be quantified, but important ones can. These latter, quantifiable uncertainties are those associated with engineering analysis (i.e., calculations), specifically the choice of appropriate soil properties, the selection of boundary and initial conditions, and the use of engineering models. Such uncertainties are important because it is on the basis of these calculations that engineering decisions are made.

As a first approximation, the uncertainties associated with the parameters that enter engineering data analysis and calculations can be separated into four groups as shown in Fig. 2.

Data Scatter

The scatter among geotechnical measurements whether made in the laboratory or in the field often exceed two or even three multiples. This scatter reflects two things: real differences of the soil and random measurement error. The former might be called spatial variability. The latter might be called 'noise.' That part of the data scatter reflecting spatial variability is the object of soil testing. That part due to noise is merely a nuisance.

Systematic Error

Systematic error is bias in an engineering prediction. That is, it affects all parts of a facility in the same way. If strength is underestimated by a 10% systematic error at one location, it would be likewise underestimated by 10% everywhere. The distinction between spatial variability and bias as sources of uncertainty is important. For example, a 10% probability of failure due to soil variability implies that one-tenth of a long embankment will perform inadequately. The same probability due to bias implies a one in ten chance that the entire embankment will perform inadequately. In geotechnical

parameter estimation, bias is caused in two ways: (1) by bias in measurement techniques and (2) by statistical estimation error due to inadequate numbers of data.

Measurement bias is common in geotechnical engineering. It is caused by such things as soil disturbance, or by differences between the way properties are measured and the way prototype structures impose load. For example, field vane measurements of undrained strength introduce a rotation of principal planes, vertical failure surfaces, and other conditions which differ from those existing under a foundation or embankment. Therefore, strengths back-calculated from embankment failures systematically differ from those measured by the field vane.

Statistical estimation bias is caused by limited numbers of measurements. From these measurements an arithmetic average is typically estimated by summing and dividing by their number. However, when another set of measurements is made, the average estimated in the same way always differs somewhat from the first. The exact numbers differ slightly and so do their averages. In addition, each of the averages differs slightly from the actual average at the site. This error is said to be 'statistical,' as it derives from the statistical variation among sets of measurements. The magnitude of statistical error can be calculated from statistical theory if the way the measurements were made is known.

Together, data scatter and systematic error constitute the uncertainty of geotechnical calculations. However, the effects of these components differ, as do the way each propagates through an engineering model.

Describing Uncertainty

Engineering data on soils properties usually vary in space and are scattered. Graphical and simple mathematical techniques are useful in summarizing this scatter so that better understanding of a set of data can be developed.

Histograms and Frequency Distributions

Histograms and frequency distributions are graphical descriptions of the variability or scatter of data.

A data histogram shows the frequency of measured values falling within specified intervals of magnitude. For example, Fig. 3 shows a histogram of standard penetration test (SPT) blow count data within a single stratum of silty alluvial sand. The intervals along the horizontal axis of the histogram are each of the same width, and the height of the bars measured along the vertical axis shows the frequency of data lying within the particular interval.

A frequency distribution is obtained by changing the vertical axis of the histogram from the frequency of data within class intervals to the cumulative fraction of data less than some particular value. The frequency distribution is a fraction (or percent)-less-than curve. Fig. 4 shows the frequency distribution for the SPT data of Fig. 3. The advantages of the frequency distribution are that it does not require data to be grouped into an arbitrary number of intervals, and that the fraction of the data less-than or greater-than any value of interest can be immediately read from the graph. The disadvantage is that the shape of the distribution of data is not as clearly apparent in a frequency distribution as it is in a histogram.

Probability paper is graph paper with special grids designed such that the cumulative frequencies of particular classes of frequency distributions plot as straight lines. Fig. 5 shows the data of Fig. 3 plotted on Normal probability paper. Normal probability grid causes bell-shaped distributions to plot as straight lines. Other types of probability paper are also available.

Mean, Standard Deviation and Coefficient of Variation

Evaluations of soil properties are conveniently expressed as a best estimate and an associated measure of uncertainty. In this report the mean and standard deviation, respectively, are used to express these two attributes.

The mean of a set of measurements x_i , $i=1, \dots, n$, is their arithmetic average,

$$\bar{x} = \frac{1}{n} \sum x_i = \text{"mean"} \quad . \quad (1)$$

This is also interchangeably called the "expected value" of x and denoted $E[x]$.

The standard deviation is the variation with respect to the mean,

$$s_x = \sqrt{\frac{1}{n-1} \sum (x_i - \bar{x})^2} = \text{"standard deviation"} \quad . \quad (2)$$

The SPT blow count data of Fig. 3 were taken in the foundation of the reregulation dam of the Carters Project. The mean of the data is $\bar{x}=8.9$ bpf and the standard deviation is $s_x=4.4$ bpf.

The standard deviation is the square root of the moment of inertia of the data about the mean. Whereas, the mean describes the "center" of the data along the X-axis, the standard deviation describes the spread. Both the mean

and standard deviation are measured in the same units as the data themselves.

In calculations it is often convenient to deal with s_x^2 rather than s_x , just as in mechanics it is convenient to deal with the moment of inertia. The square of the standard deviation is called the variance

$$V_x = s_x^2 = \text{"variance"} \quad (3)$$

and is measured in the square of the units of the data. If the data are measured in kPa, the variance is measured kPa². Given their similarity to mechanical moments, the mean and variance are often called the first and second moments of the uncertainty in an estimate. The ratio of the standard deviation to the mean, the proportional uncertainty, is called the coefficient of variation, Ω_x , and is often expressed as a percentage,

$$\Omega_x = s_x/m_x = \text{"coefficient of variation"} \quad (4)$$

Correlation

In dealing with two (or more) soil properties, not only the individual means and standard deviations may be important but also the association among different properties. For example, the undrained strength of a saturated clay is associated with water content. Thus, uncertainties in estimates of undrained strength and water content are related. The strength of such association is measured by the correlation coefficient,

$$R_{xy} = \frac{1}{n-2} \sum \left(\frac{x_i - m_x}{s_x} \right) \left(\frac{y_i - m_y}{s_y} \right) = \text{"correlation coefficient"} \quad (5)$$

R_{xy} ranges between +1 and -1. When two soil properties are proportionally associated with each other they are, on average, both simultaneously either below or above their respective means. Thus, on average, the product of $(x_i - m_x)/s_x$ and $(y_i - m_y)/s_y$ is positive, $R_{xy} > 0$. Similarly, when two properties are inversely associated, one tends to be above its mean when the other is below. Thus, the product of deviations is, on average, negative, and $R_{xy} < 0$. When two properties are not associated, $R_{xy} = 0$, and they are said to be independent. $R_{xy} = +1$ or $R_{xy} = -1$ indicates a perfect linear relation.

The correlation coefficient is a non-dimensional measure of the degree to which two parameters vary together. The two terms within the summation are the deviations of x and y measured in units of their standard deviations. That is, they are standardized deviates and are dimensionless. If the variations of x and y are not normalized by the respective standard deviations, the covariance is obtained,

$$C_{x,y} = \frac{1}{n-2} \sum (x_i - m_x)(y_i - m_y) = \text{"covariance"} , \quad (6)$$

The covariance is not dimensionless. From Eqs. 5 and 6,

$$R_{x,y} = \frac{C_{x,y}}{s_x s_y} . \quad (7)$$

Spatial Variation

Soils are geological materials formed by weathering processes and, except residual soils, transported by physical means to their present locations. They have been subject to various stress, fluid, and chemical conditions. Thus, physical properties of soil vary from place to place within a deposit.

The scatter observed in soil data comes both from this spatial variability and from errors in testing. Each of these exhibits a distinct statistical structure or signature which can be used to draw conclusions about the character of a soil deposit and about the quality of testing.

Spatial Trends in Data

In principle, the variability in a soil deposit can be characterized in detail if all the material is tested. This is not practical, however. So, rather than attempt to characterize soil properties at every point, spatial variability is divided into two parts, (i) a known deterministic trend which is characterized by a mathematical function, and (ii) a residual variability about this trend which is described statistically. This division is written,

$$x_i = t_i + u_i \quad (8)$$

in which x_i is the soil property at location i , t_i is the value of the trend at i , and u_i is the residual variation about the trend at i .

The reason u_i is characterized statistically is that there are too few data to do otherwise. This does not mean soil properties are assumed to be random, simply that their values at specific location are unknown.

Autocorrelation

The amplitude of residual variation u_i about a trend is characterized by a standard deviation or variance. By the procedure through which a trend is estimated, the residuals about it have mean zero. In addition to amplitude, one other attribute of the residual variation is important. This is their 'waviness' or dominant spatial frequency.

This waviness of residual soil data reflects a spatial structure in addition to that characterized by the trend. For example, if strength measurement, say, at depth i in a profile lies above the average trend of strength with depth, as a general rule strength measurements immediately above and below in the same boring also lie above the trend, or vice versa. This association of nearby data is called 'autocorrelation'. The further away the association exists, that is the longer the apparent 'wave length' of the residuals, the greater the autocorrelation. More formally, autocorrelation is the property that residuals off the mean trend are not statistically independent, and that the degree of correlation among them (i.e., as measured by R_{xy} of Eqn. 5) depends on their relative separation in space.

The effect of correlation structure on residual variation can be seen in Fig. 6 in which four cases are sketched schematically. Spatial variability about a trend is characterized by a standard deviation or variance and by autocorrelation. Large variance implies that the absolute magnitude of the residuals is large; large autocorrelation implies that the 'wave length' of variation is long.

If x is a soil property within a soil deposit that is zonally homogeneous, then at locations close together the residuals $u_i = x_i - t_i$ and $u_j = x_j - t_j$ should be expected to be similar. As their separation $|i-j|$ approaches zero, u_i and u_j become the same term. Conversely, at locations widely separated, the residuals should not be expected to be similar. This spatial association between residuals off the trend can be summarized by a function describing the similarity of u_i and u_j as the distance $\delta = |i-j|$ increases. This function is called the autocovariance function,

$$C_x(\delta) = \left(\frac{1}{n-2}\right) \sum (x_i - t_i)(x_{i+\delta} - t_{i+\delta}) \quad . \quad (9)$$

The autocovariance function expresses the covariance of two residuals off the trend as a function of their separation distance. If $C(\delta)$ is normalized by the variance of the residuals V_u , the autocorrelation function is obtained,

$$R(\delta) = \frac{C(\delta)}{V_u} \quad . \quad (10)$$

Figure 7 shows the autocovariance function estimated for the SPT data in borings at the reregulation dam at the Carters Project. The extent of horizontal correlation in these data is about 500ft (167m).

It is important to emphasize that the autocovariance function is an artifact of the way soil variability is modelled. The autocovariance depends on how a 'trend' is separated from residuals. Since there is nothing innate about the chosen trend t_i , and since changing the trend necessarily changes $C(\delta)$, the autocovariance function reflects a modeling decision. The soil properties are not assumed to be 'random,' rather their non-random but unknown spatial variability is characterized by mathematics which are similar to those used in random process theory.

Size Effect Factor

The volume of soil influenced by an in situ test or contained in a laboratory specimen is small compared with that influenced by a prototype structure. To make predictions of how the prototype will perform, estimates need to be made of the properties within these larger representative volumes, and of the variability among the properties of the representative volumes.

This is done by assuming the representative volume to be composed of a large number of elements each of small size (e.g., the size of a test specimen). The mean and standard deviations of specimen sized elements are found from the formulas above, then using the autocorrelation function as a description of spatial structure, a mean and standard deviation for the larger volumes is be calculated. These calculations are summarized in a size-effect factor, R , which in many cases can be expressed by simple formulas or can be tabulated.

Empirically, the variability of soil properties among small elements is larger than that among large elements. Within a small volume physical properties tend to be more or less uniform. Some individual elements may have greater than average strength, say, while some may have less than average. Within each element, however, there is less variability than there is among the average properties of different elements. Within large volumes of soil the opposite is true, there tends to be a mixture of high and low values. Thus, with small volumes the properties of individual elements vary sharply from the mean across a site, but with large volumes internal variations balance out such that the average properties from one large element to another differ little. The mean of large volumes remains the same as the mean of small volumes, but the standard deviation of the average property from one large volume element to the next is smaller than the standard deviation of the average property from one small volume element to the next.

The extent of averaging of properties within a large volume of soil depends on the structure of the spatial variation. More precisely, the extent of averaging depends on the standard deviation of properties from point to point and on the autocorrelation function.

As an example of spatial averaging, consider the variability of average SPT blow count among borings in a homogeneous soil. Fig. 8 shows six boring logs. If one N value from each boring is randomly chosen and the standard deviation among them calculated, some value for s_N is obtained, here 2.5. If two N values in each boring are chosen, and the average taken, then again the standard deviation among boring averages is calculated. From this calculation a smaller standard deviation results. Note that the values shown in Figure 9 are one possible set and that depending on which N values are selected the values for m_{3N} and s_N will change somewhat. Continuing, the greater the number of N values included in the average for each boring, the smaller the standard deviation of the boring-averaged N across the six borings. Spatial variations increasingly average out as the volume of soil or rock within the element considered increases. The reduction of variability for a continuously varying formation is illustrated graphically in Fig. 9. The y-axis shows the size effect factor R , the ratio of the variance of average element properties to the variance of point properties for a two-dimensional element. The x-axis shows the normalized size of the element as a function of the autocorrelation distance.

By a similar token, if extreme local properties in a volume of soil are important for engineering performance, then rather than becoming less important as volume increases spatial variations become more important. Conditions become progressively worse as volume increases. For example, if performance depends on the least dense part of a compacted clay core or on the least favorably oriented discontinuity in a rock mass, then the larger the volume of soil or rock, on average the worse the most extreme element. So, spatial

variations may work either for or against engineering performance depending on the problem at hand.

Statistical Methods in Dam Projects

The project identified four major activities of geotechnical engineering aspects of new dam projects to which the application of statistical methods might provide benefit over present procedures. These are:

1. Site characterization planning and data analysis.
2. Analyzing the reliability of engineering performance prediction (i.e., modeling activities).
3. Quality control and quality assurance of construction operations.
4. Design of instrumentation networks and the analysis of performance-monitoring data.

Most problems dealt with in the literature of statistical and probabilistic methods for geotechnical engineering can be grouped under at least one of these four headings. However, the extent of coverage varies considerably across the four areas.

Several authors have reported applications of statistical methods to geotechnical engineering problems similar or related to those encountered on new dam projects. This literature is too extensive to be summarized briefly. This section references a small part of that literature pertinent to new dam projects.

Site Characterization

A number of authors have addressed statistical aspects of site characterization. Predominantly these articles deal with one or more of the following issues, (a) choice of numbers and locations of borings or tests, (b) interpola-

tion or mapping of measurements among boring or test locations, (c) estimation of design parameters from test data, (d) search for anomalous details of site geology. Pertinent references include:

Alonso, E.E. and R.J. Krizek (1975): "Stochastic formulation of soil properties," 2nd ICASP*, Aachen: 9-37.

Baecher, G.B. (1978): "Analyzing Exploration Strategies, in: C.H. Dowding (ed.), Site Characterization and Exploration, ASCE/NSF.

Bogardi, I., A. Bardossy, and L. Duckstein (1983): "Geostatistics for the estimation of soil parameters and for observation network thereof," ICASP4, Florence.

Brown, A.A., (1960): "Search Theory and Problems of Exploration", Penn State University, Minerals Ind. Exp. Sta., Bull., 72: 33-37.

Christian, J.T., and W.F. Swiger (1975): "Statistics of liquefaction and SPT results" Proceedings ASCE, 101(GT11): 1135-50 (with discussion).

Dowding, C.H., (1976): "Comparison of Predicted and Encountered Geology of Seven Colorado Tunnels," Proc. of 1976 Rapid Excavation Technology Conference, Las Vegas, Nevada.

Fardis, M., and Veneziano, D., (1981): "Estimation of SPT-N and Relative Density," Journal of the Geotechnical Engineering Division, Proc. ASCE, V.107(GT10).

Grant, R., (1973): "A Probabilistic Approach to the Search for Solution Cavities in Limestone," SM Thesis, MIT.

Grayson, C.J., (1960): Decisions Under Uncertainty: Drilling Decisions by Oil and Gas Operators. Graduate School of Business Administration, Harvard.

Kay, J.N., (1976): "Safety factor evaluation for single piles in sand" Proceedings ASCE, 102 (GT10): discussion Meyerhof 103(GT4): 362, 104(GT1).

Kotzias, P.C., and A.C. Stamatopoulos, (1983): "Graphic statistics for multiple geotechnical data," in ICASP4, Florence.

Lumb, P., (1966): "The variability of natural soils," Canadian Geotechnical Journal, 3(2): 74-97.

*ICASP = International Conference on Applications of Statistics and Probability to Soil and Structural Engineering.

McGuffey, V., Iori, J., Kyfor, Z., and Grivas, A.D., (1981): "Statistical Geotechnical Properties of Lockport Clay," Transportation Research Board No. 809, National Academy of Science, pp. 54-60.

Schultze, E., (1971): "Frequency distributions and correlation of soil properties," 1st ICASP, Hong Kong: 371-387.

Engineering Analysis

The overwhelming majority of articles published on the use of statistical methodology for geotechnical engineering focus on performance predictions and design decisions. Primarily this work deals with, (a) derivation of uncertainty in performance prediction from uncertainty in input parameters, (b) optimization of design in the face of uncertainties about performance and a well-defined objective function for facility performance. Pertinent references include:

Alonso, E.E. (1976): "Risk analysis of slopes and its application to slopes in Canadian sensitive clays," *Geotechnique*, 26(3): 453-472.

Athanasίου-Grivas, D., and Asaoka, A., (1982): "Slope stability prediction under static and seismic loads," *Journal of the Geotechnical Engineering Division, ASCE*, Vol. 108, No. GT5, May.

Biernatowski, K., and W. Brzakala (1983): "A stochastic model of subsoil deformations," in ICASP4, Florence.

Bowles, D.S., L.R. Anderson, and R.V. Canfield, (1978): "A systems approach to risk analysis for an earth dam," *International Symposium on Risk and Reliability in Water Resources*, Waterloo, Ontario, Canada.

Call, R.D., and D.E. Nicholas (1978): "Prediction of step path failure geometry for slope stability analysis," *Proc., 19th U.S. Symp. on Rock Mechanics*.

Cambou, B. (1978): "Applications of first-order uncertainty analysis in the finite elements method in linear elasticity" *Proc. 2nd ICASP, Aachen*.

Christian, J.T., (1980): "Probabilistic soil dynamics: State-of-the-Art" *Jour. of the Geotechnical Engineering Division, ASCE* v106(GT4): 385-397.

Cornell, C.A. (1971): "First Order uncertainty analysis of soils deformation and stability," 1st ICASP, Hong Kong: 130-144.

D'Andrea, R., and Sangrey, D.A., (1982): "Safety factors for probabilistic slope design," Journal of the Geotechnical Engineering Division, ASCE, Vol. 108, No. GT9, Sept., pp. 1101-1118.

Polayan, J.I., Hoeg, K., and Benjamin, J., (1970): "Decision theory applied to settlement predictions," Journal of Soil Mechanics and Foundations Div., ASCE., V. 106 (SM7): 1127 ff.

Harr, M., (1977): Mechanics of particulate media -- A probabilistic approach. McGraw-Hill NY.

Hoeg, K., and R.P. Murarka (1974): "Probabilistic analysis and design of a retaining wall", Proceedings ASCE, 100 (GT3).

Hynes, M.E., (1976): "Reliability analysis of earth slopes: I-95 Embankment Case Study," Master of Science Thesis at M.I.T., Supervised by E.H. Vanmarcke, May.

Kraft, L.M. (1968): "Probability Application in Soil Engineering," Ph.D. Thesis Ohio State U. 85pp.

Matsuo, M., and Kuroda, K., (1974): "Probabilistic Approach to Design of Embankments," Soils and Foundations, Japanese Society of Soil Mech. and Found. Engineering, Vol. 14, No.2, June.

Tang, W.H., Yucemen, M.S., and Ang, A.H-S., (1976): "Probability-based Short Term Design of Soil Slopes," J. Canadian Geot. Soc., Vol. 13, pp. 201-215.

Wu, T.H., (1974): "Uncertainty, safety, and decision in soil engineering," Journal of the Geotechnical Engineering Division, ASCE, Vol. 100, No. GT3, paper 10434, pp.329-348.

Construction Inspection

Despite the obvious applicability of statistical methods to quality control and quality assurance for earthwork construction relatively few detailed treatments have been published. Many of these apply to highway or airfield construction, but face problems similar to those in earthdam construction. The most noteworthy reference on statistical QC for dam projects is Kotzias and Stamotopoulos (1975), and for highways is Kuhn (1972). Pertinent references include:

Blaut, H., (1975): "Application of statistics for quality control of soil-cement stabilization in highway construction," ICASP2 Aachen, F.R. Germany.

Fang, H.Y., (1975): "Sampling plans and construction control," Proceedings of 2nd Inter. Conf. on Applications of Stat. & Prob. in Soil and Struct. Eng., Aachen, F.R. Germany.

Ingles, O.G., (1971): "Statistical control in pavement design" Proc. 1st ICASP, pp. 267-312 Hong Kong.

Kotzias, P.C., and A.C. Stamotopoulos (1975): "Statistical quality control at Kastraki earth dam," Proceedings ASCE, 101(GT 9): 837-54 discussion: Covmavlos, J.D. 102 (GT1):815, Sherard. J.L. 102 (GT1): 816-7.

Kuhn, S.H., (1972): "Quality Control in Highway Construction," National Institute of Road Research, South Africa, Bulletin No. 294.

Kuroda, K., and Kanada, Y., (1983): "New quality control method and its application to construction of compacted sand pile," in ICASP4, Florence.

Performance Monitoring

Few papers in the geotechnical literature explicitly address instrumentation as methodologically distinct from site characterization problems. Those that do, focus more on observational approaches in general rather than just the design of instrumentation networks. Pertinent references include:

Ashley, D.B., Veneziano, D., Einstein, H.H., Chan, M.H. (1981): Geologic prediction and updating in tunneling - A probabilistic approach. Proceedings, 22nd U.S. Symposium on Rock Mechanics, M.I.T.

Hachich, W.C., (1981): "Seepage Related Reliability of Embankment Dams," MIT PhD Thesis Dept. of Civil. Eng.

Matsuo, M., and Asaoka, A., (1978): "Dynamic Design Philosophy of Soils Based on the Bayesian Reliability Prediction," Soils and Foundations, Vol.18, No.4., pp.1-17.

Current Directions of Research and Development

Current research on statistical methods for geotechnical engineering is of interest in that it suggests the sorts of technology that might be available in the near future. In large part current work foccuses on four topics:

1. System reliability for large projects.
2. Analysis of spatial variation.
3. Risk assessment for waste disposal sites.
4. Artificial intelligence for data analysis.

Systems Reliability

Work on system reliability for large systems involves multiple modes of failure, interactions among failure modes, and the analysis of design alternatives for maximizing the overall reliability of large facilities. Typical of these large systems are refineries on poor soil conditions or in seismically active regions, water retaining structures such as tailings dams which may lose containment in many ways, and nuclear waste repositories. The analysis of such facilities usually involves fault or event trees or large-scale Monte Carlo simulations.

Large-scale systems reliability analyses tend at present to be expensive of man-power and computer time. They are also poorly tested on actual projects. Should risk assessment of new dam projects at some time in the future become necessary, systems reliability techniques would probably form an integral part of such assessments. For the present, however, the practical and routine use of systems reliability techniques for new Corps projects appears far off.

Analysis of Spatial Variation

Improved techniques for analyzing soil and rock data to estimate the structure of spatial variations are actively under development at several places. This work involves new mathematical procedures for analyzing spatially defined data in order to overcome limitations of present methods for estimating trends and autocovariance (or variogram) functions.

This work appears to have immediate benefit to the analysis of geotechnical data on new dams projects, and to the implementation of automated data processing. Its primary benefit lies in replacing current procedures which are known to be biased and inefficient by statistically optimal procedures, (i.e., procedures which make better use of data and give less variable estimates). Certain application of this technology have been made to the Carters Project data base and are reported in Parts III and IV of this report.

Risk Assessment of Waste Sites

A great deal of effort is currently being expended on assessing risks associated with toxic waste sites, particularly high-level nuclear waste. This work is driven by regulatory concerns about the long term safety of waste storage facilities, at the Nuclear Regulatory Commission (NRC), the Environmental Protection Agency (EPA), and elsewhere. Regulation standards and criteria already published by these agencies are based on probabilistically quantified risks associated with geotechnical performance (e.g., 10CFR60, EPA draft standard 9 on high level waste). In the immediate future such efforts will probably extend to chemical waste sites as well.

The spin-off of this large amount of work on risk assessment for geotechnical engineering aspects of toxic waste facilities to new dam projects

will be twofold. First, a great deal of experience will be developed in exercising statistical techniques on real projects. This experience will help identify and refine statistical procedures which will have equal applicability to dams as to waste facilities. Second, methodology will be developed and tested for performing practical probabilistic risk analyses on prototype facilities. If future needs develop for risk assessment studies of dam projects, much of the methodology developed in the waste program will be directly transferable.

Artificial Intelligence for Data Analysis

Considerable interest has been generated in the past few years in the application of artificial intelligence technology to geotechnical engineering, especially to data analysis and the interpretation of in situ test data. Knowledge based expert systems (KBES) have been applied to interpreting cone penetration profiles (Mularky, 1984), and statistical pattern recognition has been applied to reconstructing soil profiles from SPT data (Erikson, 1985). In essence, AI techniques bring advanced programming technology to problems of analyzing data. The advantage of AI techniques over algorithmic programs is that they allow interpretation rather than just analysis of data.

The importance of AI technology for geotechnical engineering aspects of dam projects, especially on problems of data analysis, the selection of engineering parameters, and numerical modeling of embankment or foundation performance is argued by its proponents to be substantial. It is argued that AI technology may reduce the number of routine judgemental decisions that go into data analysis or modeling, and will thus streamline these phases of design. The technology will also further the trend of automated data

processing by allowing more operations to be programmed. Certain application of this technology to the Carters data set have been made in the context of the present project and are reported in Parts III and IV of this report.

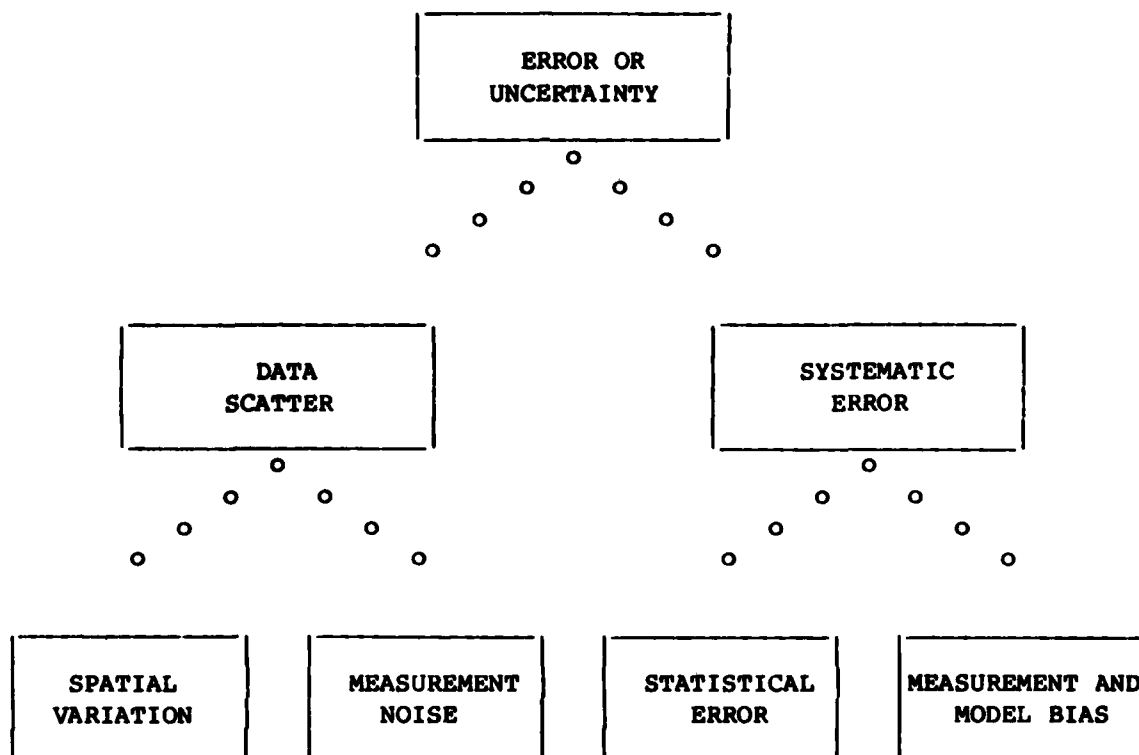
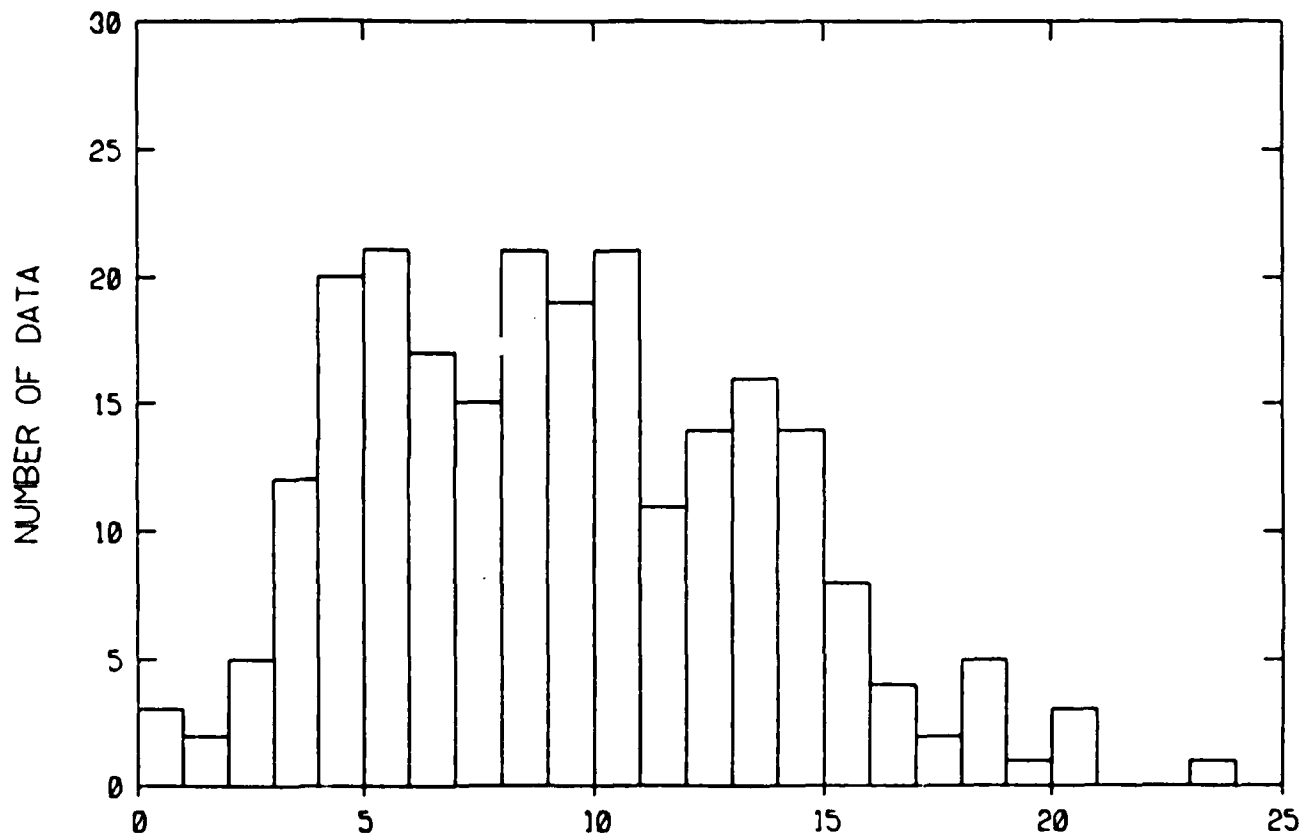


Figure 2. Sources of Error or Uncertainty
in Soil Property Estimates

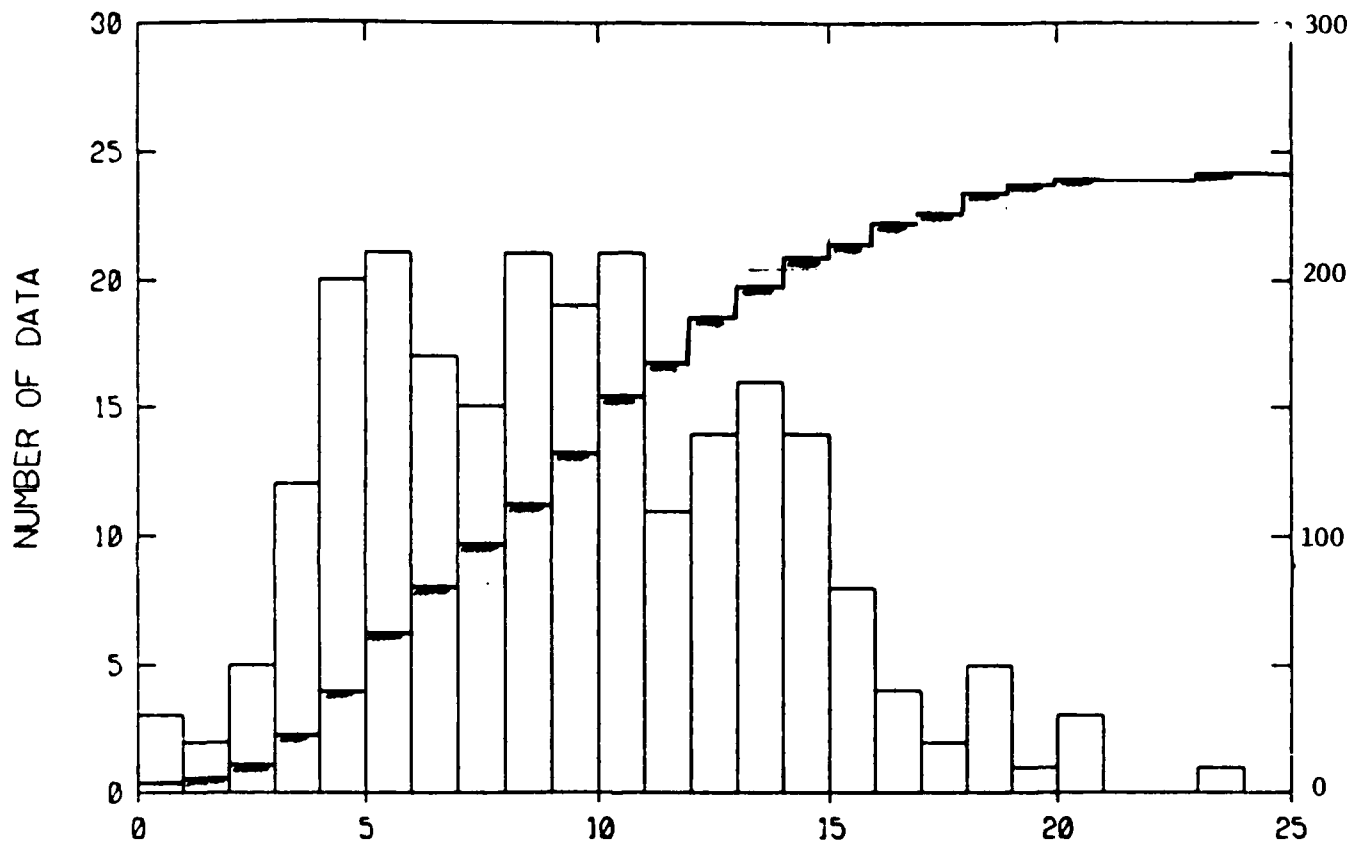
TOTAL NUMBER OF DATA : 235
 MEAN : 8.9021 STD DEV : 4.4283
 NUMBER OF OUTLIERS BEYOND LOWER LIMIT OF PLOT : 0
 NUMBER OF OUTLIERS BEYOND UPPER LIMIT OF PLOT : 0



N VALUES - UNCORRECTED
 RE-REG DAM STA 25+00 - STA 32+00

Figure 3. Histogram of Standard Penetration Test Blow Counts from Silty Sand.

TOTAL NUMBER OF DATA : 235
 MEAN : 8.9021 STD DEV : 4.4283
 NUMBER OF OUTLIERS BEYOND LOWER LIMIT OF PLOT : 0
 NUMBER OF OUTLIERS BEYOND UPPER LIMIT OF PLOT : 0



N VALUES - UNCORRECTED
 RE-REG DAM STA 25+00 - STA 32+00

Figure 4. Cumulative Frequency Distribution of the SPT Data in Figure 3.

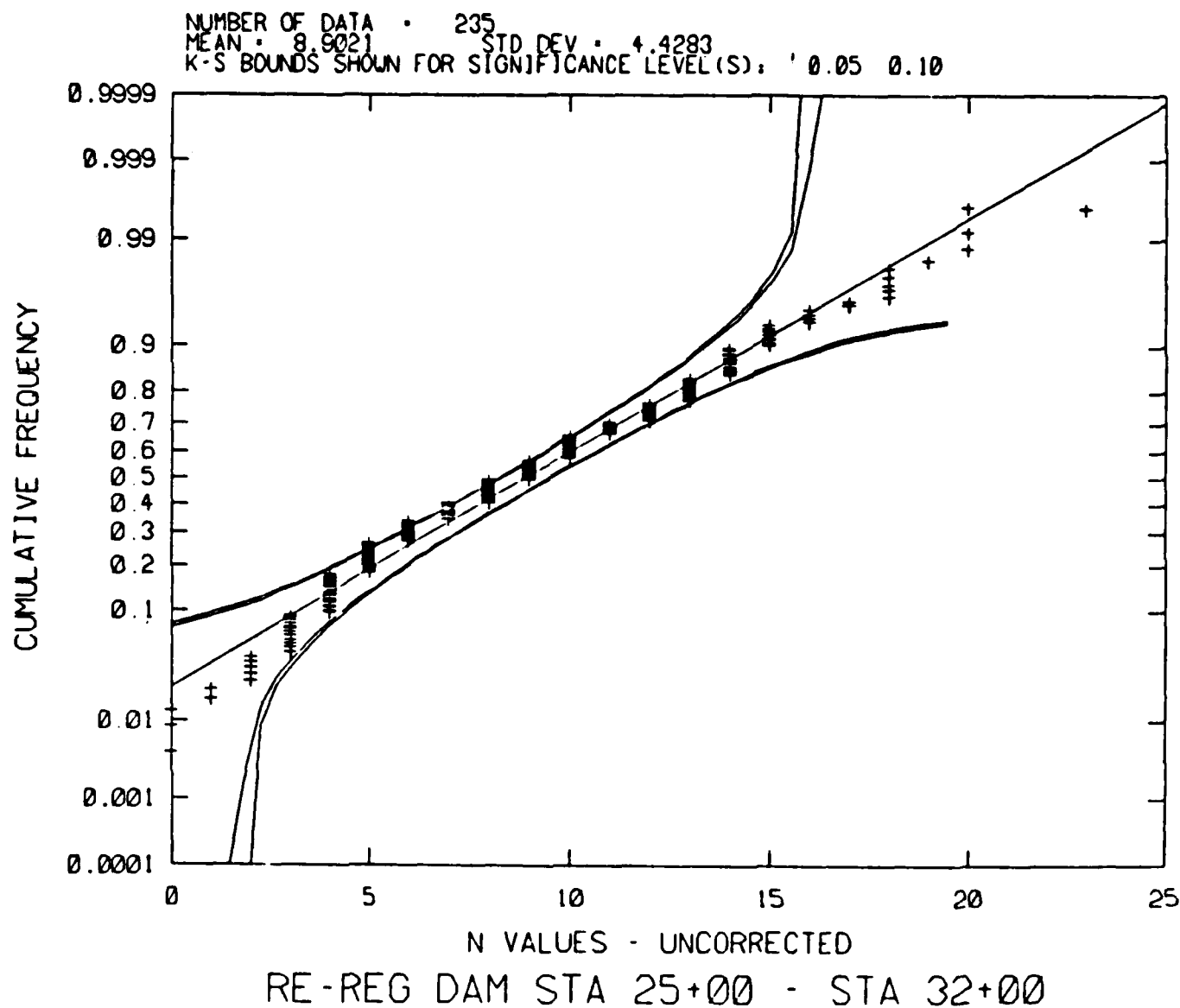


Figure 5. Normal Probability Paper Plot of the Data of Figure 3.

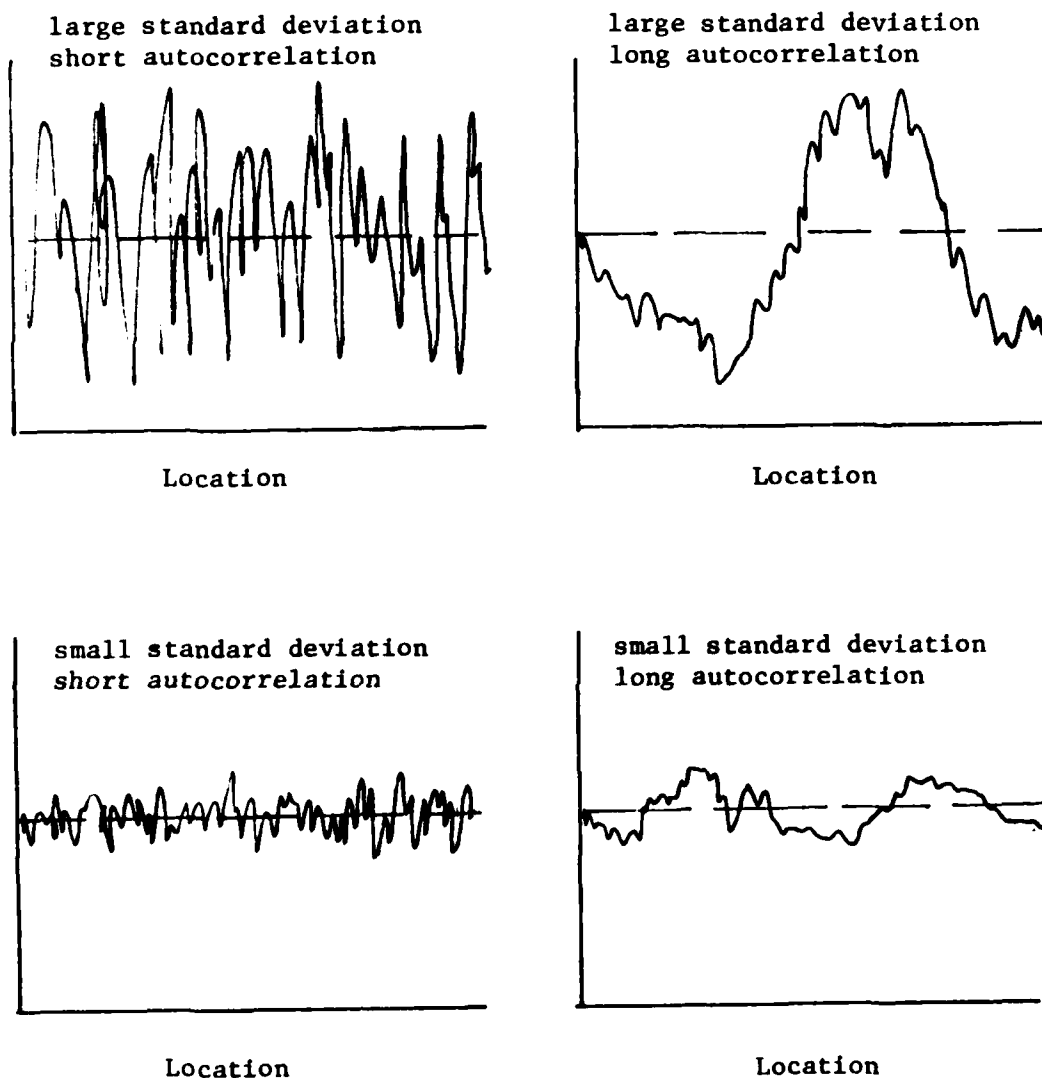


Figure 6. Effect of Correlation Structure on Spatial Variability.

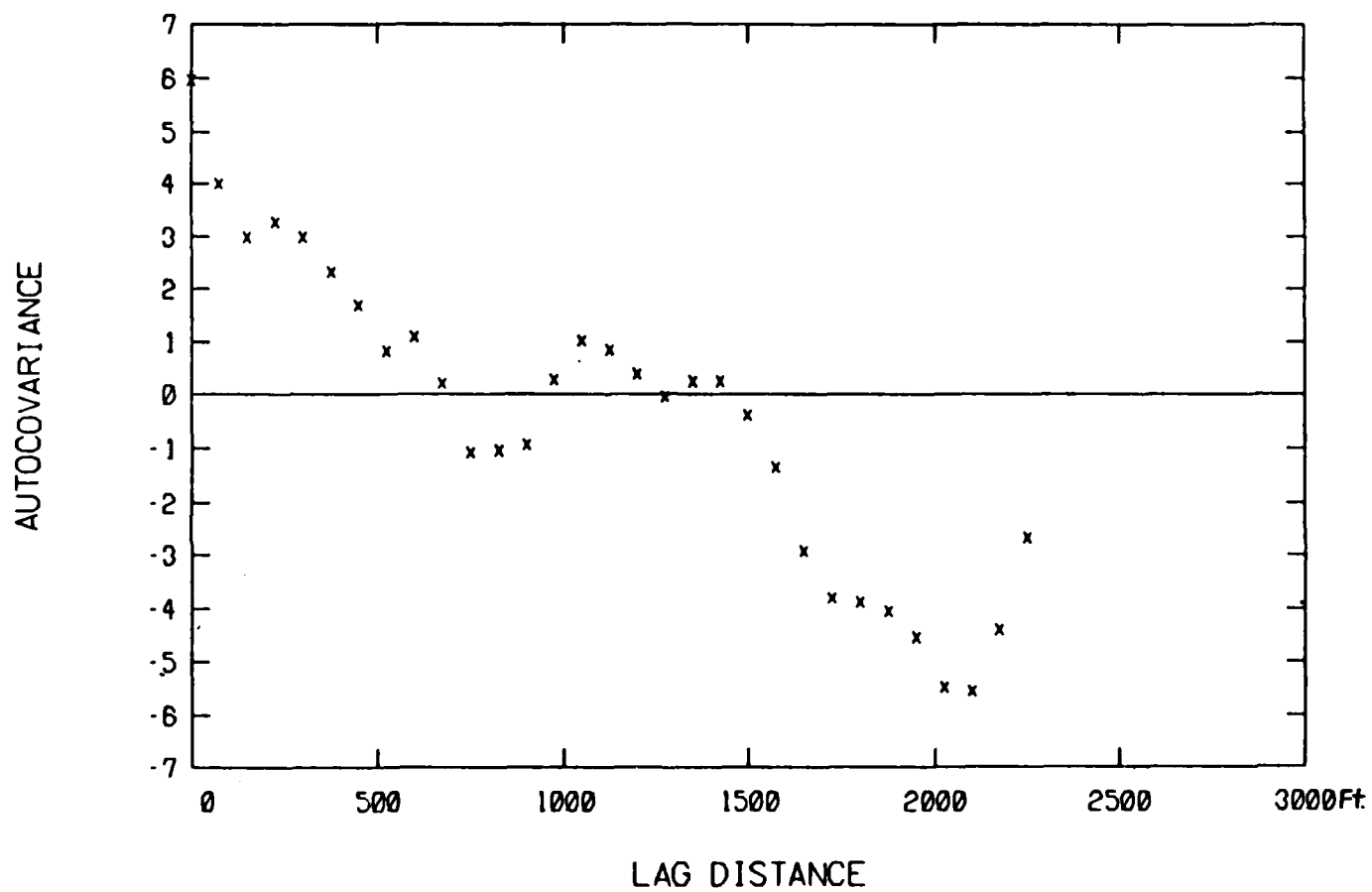


Figure 7. Autocorrelation Function for SPT Data at Reregulation Dam

	BORING #	1	2	3	4	5	6
DEPTH		-----					
1		2	1	2	8	3	4
2		3	8	5	3	7	4
3		8	6	5	3	7	5
4		6	6	7	0	8	7
5		0	2	5	2	5	0
6		3	2	4	1	9	4
7		3	5	0	0	4	1
8		8	3	0	8	8	7

Figure 8. Six Hypothetical Boring Logs Showing SPT Data

Number of data	Mean of Boring Averages and	Standard Deviation Among Averages
-------------------	--------------------------------	--------------------------------------

$n = 1$	$m_N = 3.3$	$s_m = 2.5$
---------	-------------	-------------

$n = 2$	$m_N = 5.7$	$s_m = 2.0$
---------	-------------	-------------

$n = 8$	$m_N = 4.2$	$s_m = 0.9$
---------	-------------	-------------

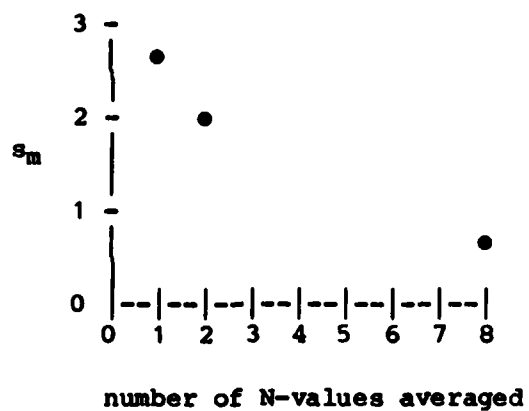


Figure 9. Reduction of Variance Among Boring Averages as Number of Data in Each Average Increases.

PART III: CARTERS PROJECT APPLICATIONS

Part III summarizes the process of planning, design, and construction of the Carters Project. The purpose of this summary is to identify major decision points, outline the uncertainties which affected the decisions, and consider the data which were collected to deal with the uncertainties.

Description of Carters Project

The Carters Dam Project is part of the Alabama-Coosa River and tributaries development. The Dam Site is located 26.8 miles above the mouth of the Coosawattee River near Carters in northwest Georgia. It is 75 miles from Atlanta, Georgia and 50 miles southeast of Chattanooga, Tennessee. The drainage basin above this project has an area of 376 square miles with a length of 50 miles and a maximum width of 28 miles (Figure 10).

An overall description of the Carters Project is presented in Design Memorandum 5, "General Design." The dam has a height of 447', length of 1950' and radius of 2100'. It has a maximum power pool at elevation 1072, and a minimum at elevation 1022. The main dam is a zoned, rolled rockfill embankment with a centrally located impervious core (Figure 11). The principal appurtenant structures are three saddle dikes on the left bank of the reservoir, a head race channel with intake structures excavated in rock in the right bank, and a powerhouse located 200' below the downstream toe of the dam on the right bank. A reregulation dam is downstream of the main dam (mile 25). The general site layout is shown in Figure 12.

Construction at the Dam site began in April 1962. Excavation of the diversion tunnel began in June 1963 and completed in January 1964. Construction of the Main Dam (Phase I) began in April 1964; Phase II in April 1965. Phases I and II were completed by February 1970. The reregulation dam construction began in October 1970 and completed in May 1974.

Geology of the Carters Site

The Carters Project site lies astride one of the major thrust features of the southeastern portion of the United States, the Cartersville Fault. This feature separates the Piedmont Physiographic Province to the east from the Valley Physiographic Province to the west, and causes a sharp drop in elevation from which the head drop at the site benefits.

The main embankment, powerhouse and intake structures, and diversion tunnel all lie within the hard (700 to 3000 psi unconfined compression strength) metasedimentary formations of the Piedmont (quartzite, phyllite, and argillite). The reregulation dam lies atop more recent sedimentary rocks in Valley Province (limestones, shales, and sandstones).

At the main embankment site, alluvial sands and gravels occur in the river valley bottom, and dense red to yellow clay from weathered quartzite and phyllite overlie the parent rock along the valley walls. The deepest weathering to about 60' occurs on the right abutment. In the Valley Province below the main embankment surficial alluvial silts and sands with some clays occur to a depth of about 50'.

Core materials for the main embankment came in part from weathered rock at the main embankment site, and in part from overburden in the Valley Province

below. Rock fill for the main embankment came primarily from rock excavations at the head race, diversion tunnel, and downstream borrow quarries. Fill materials for the reregulation dam came primarily from overburden in the Valley Province.

Chronology of Design and Data Collection

For the purposes of the present study a chronology of design phases, decisions, and data collection activities for the Carters Project was compiled in Table 1. Feasibility studies began in 1957 and continued through site selection in March 1961. Test fills of core and shell materials were undertaken in the spring of 1961, and a conceptual design was prepared in 1962. Professor Arthur Casagrande was principal consultant to the Project throughout this period. The preliminary design was completed by July 1963. Detailed design, further test fills and site characterization were undertaken in 1963 and 1964 while construction at the site was under way. The detailed review for the main embankment was completed in October 1964. A detailed summary of the project, design, and construction is given in the Embankment Criteria and Performance Report of 1974.

The main data collection phases for geotechnical purposes consisted of three stages of borings, topographic mapping and geological characterization, and three stages of test fills (Table 2). Information from the boring programs at the main embankment primarily used for identifying the extent and depth of weathering in foundation rocks.

Core recoveries at the main embankment site were better than 90% in partially weathered rock, and generally better than 98% in sound rock. In all, approximately 235 NX-sized core borings were made for the main embankment and adjacent structures. Additional borings were made at potential borrow sources and along planned road right-of-ways. Approximately, 63 borings were made at the site of the reregulation dam.

The purpose of the test fills was to evaluate the suitability of rock fill materials for the main embankment, and to test the efficacy of compaction equipment and procedures. The test fills were an important source of engineering information for design decisions and for the specification of construction equipment.

Design conferences on Carters Dam were held in September 1962, September 1963, November 1963, and July 1964. Professor A. Casagrande, the principal geotechnical consultant on the project, was present at all but the second meeting. The design conferences led to major decisions or assumptions for design, as noted in Table 1.

Post-Filling Performance

Carters Dam was extensively instrumented during construction and these instruments have been continuously monitored since the Dam was placed in operation. The main embankment was instrumented to determine horizontal and vertical displacements internal to the embankment, and total stress and pore pressures. A schematic of the instrumentation network is shown in Figure 13. The main embankment instrumentation includes Idel displacement gages, Gloetzl hydraulic soil and pore pressure cells, an Eastman Inclinator, and

specifically designed water level devices. The instrumentation is described in Carters Dam, Design Memorandum No. 26, "INSTRUMENTATION," dated October 1970.

No unusual post-filling performance has been observed. Periodic Inspection Reports have noted that the conditions of all components of the project appear good, with no evidence of structural distress. Seepage at the toe of the main dam has remained minimal.

Table 1. Chronology of Major Design Decisions and Data Collection Activities

DATE	PROJECT STAGE	ACTIVITY	DATA COLLECTED	IMPORTANT DECISIONS	IMPORTANT ASSUMPTIONS	SOIL PARAMETERS NEEDED	SIGNIFICANT UNCERTAINTIES
1957	FEASIBILITY		6 BORINGS & DETAILED TOPOGRAPHY OF 270 ACRES	JUSTIFIED THE CONSTRUCTION OF A DAM ALONG THE COOSAWATER RIVER			
MARCH 1961	FEASIBILITY	SITE SELECTION REPORT	TOPOGRAPHIC SURVEY: 50 ACRES @ CARTERS SITE 200 ACRES @ GOBLE SITE 5 @ CARTERS SITE 9 @ GOBLE SITE	BASED ON ECONOMIC AND GEOLOGICAL CONSIDERATIONS - "PLAN 1" ADOPTED "PLAN 1" CONSIST OF A SINGLE ROCK FILL DAM AT THE CARTERS SITE WITH ALL PRINCIPLE FEATURES ON THE LEFT BANK (POWER PLANT FREE OVERFLOW SPILLWAY, SERVICE TUNNEL)	GOBLE SITE NOT ECONOMIC DUE TO DEEP WEATHERING AT CARTERS SITE; JOINTING IN CORES INDICATE "NORMAL GROUTING PROGRAM" SUITABLE FILL CAN BE OBTAINED FROM TOPS OF ADJOINING RIDGES		QUANTITIES OF WEATHERED ROCK MATERIAL - IN SITU
LETTER OF MAY 1961	FEASIBILITY					MORE INVESTIGATION NEEDED TO DETERMINE SUITABILITY OF ROCK ROCKFILL MATERIAL	
OCTOBER 1961	SURVEY	TEST FILLS				FIRST CONTRACT TO INCLUDE THE CONSTRUCTION OF 4 TEST FILLS TO DETERMINE THE SOURCE OF ROCK FOR DAM	
CONFERENCE WITH THE CASAGRANDE 27 SEPT. 1962	PRELIMINARY DESIGN	ALTERNATE IMPROVEMENT PLAN		IMPORTANT PLAN SELECTED INCLUDES GATED SPILLWAY (AS OPPOSED TO FREE OVERFLOW SPILLWAY) POWERHOUSE ON RIGHT BANK, DIVERSION TUNNEL ON LEFT BANK, SERVICE NOT REQUIRED	MAJOR FAULT LIES TOO CLOSE TO EXISTING POWERHOUSE SITE EXCAVATION INTO AREA BEYOND RIGHT ABUTMENT WILL YIELD A LARGE QUANTITY OF HIGHLY WEATHERED ROCK WHICH MAY NOT BE SUITABLE FOR USE IN DAM		QUANTITIES OF WEATHERED ROCK MATERIAL
LAB TRIAXIAL TESTING OF TEST FILLS PERFORMED BETWEEN NOV. 62 AND OCT. 63		TEST FILLS		SUITABILITY OF VARIOUS MATERIALS, LIFT THICKNESSES & TYPE OF COMPACTION EQUIPMENT FOR THE VARIOUS ZONES OF DAM WERE DECIDED		TRIAxIAL TESTS ON ALL MATERIALS	
		FREEBOARD		FREEBOARD INCREASED FROM 3' TO 5'	CHANGE IN FREEBOARD DUE TO "LARGE SIZE OF PROJECT & NATURE OF MATERIALS TO BE USED"		

DATE	PROJECT STAGE	ACTIVITY	DATA COLLECTED	IMPORTANT DECISIONS	IMPORTANT ASSUMPTIONS	SOIL PARAMETERS NEEDED	SIGNIFICANT UNCERTAINTIES
JAN. 1963	SURVEY	BORINGS		DIVERSION TUNNEL, UPSTREAM & DOWNSTREAM CARTER DAM IMPLEMENTED TO PROTECT CONSTRUCTION AREA UPON COMPLETION: DIVERSION TUNNEL-CONCRETE PLUG TO BE INSERTED UPSTREAM CARTER DAM BECOMES OFFSPRANG OF MAIN DAM ROCK FILL DAM WITH 3 SADDLE DICES, SEEPAGE CONTROL VIA GROUT CURTAIN AND CORE TRENCH EXCAVATED TRENCH AND ROCK		ADDITIONAL GEOLOGICAL INVESTIGATION FOR DATA TO DETERMINE FINAL LOCATION OF DIVERSION TUNNEL & OTHER STRUCTURES	
12 JULY 1963	PRELIMINARY DESIGN	HYDRAULIC DESIGN					
CONFERENCE OF 1 SEPT. 1963	PRELIMINARY DESIGN	GENERAL PLAN	TOPOGRAPHIC SURVEY - 300 ACRES 160 CORE BORINGS		USE DATA FROM COUGAR DAM FOR INDICATIONS OF DENSITIES TO BE USED IN TESTING MATERIAL	ADDITIONAL TRIAXIAL TESTING ON TESTFILL AND "COMFIELD" MATERIAL	CORE MATERIAL TO BE USED - "COMFIELD" OR TESTFILL
LETTER OF SEPT. 1963	DETAILED DESIGN	SUPPLEMENTARY INFORMATION		ROCK BEAMS ABOVE POWERHOUSE CHANGED FROM 30" TO 50" TO "PROVIDE AN ADDITIONAL MARGIN OF SAFETY IN CASE OF FUTURE ROCK FALLS"			
CONFERENCE WITH ARTHUR CASAGRANDE 14 NOV. 1963	DETAILED DESIGN	DESIGN PARAMETERS		SET SPECIFICATIONS ON LIFT SIZE, MATERIAL TYPE AND MACHINERY TO BE USED FOR ZONES I-III	TENTATIVELY BLT SLOPES FOR ROCK FILL MATERIAL PROVIDING FURTHER STABILITY ANALYSIS IN II SET FOR OUTER UPSTREAM & DOWNSTREAM SLOPES		
	DETAILED DESIGN	ROCK BEAMS					
	DETAILED DESIGN	TEST FILLS					

DATE	PROJECT STAGE	ACTIVITY	DATA COLLECTED	IMPORTANT DECISIONS	IMPORTANT ASSUMPTIONS	SOIL PARAMETERS NEEDED	SIGNIFICANT UNCERTAINTIES
		CORE MATERIAL		"COMFIELD" MATERIAL NOT TO BE USED FOR CORE-TO-SILTY MAY REQUIRE SAND & GRAVEL FOR FILTER $\frac{1}{2}$ IN ABOVE WORK TEST FILL $\frac{1}{4}$ " MATERIAL TO BE USED FOR CORE INSTEAD	DR. A. CASAGRANDE SUGGESTED ϕ VALUES FOR USE IN THE STABILITY ANALYSIS FOR WEDGE ANALYSIS ϕ ALONG BASE OF NEUTRAL BLOCK - ORIGINALLY FULL VALUE OF 39.5' USED NOW SUGGESTED THAT A LOWER VALUE OF 30' BE USED	ϕ ALONG BASE OF NEUTRAL BLOCK	ϕ VALUES WITH EMBANKMENT AS WELL ϕ VALUE BETWEEN EMBANKMENT AND FOUNDATION
CONFERENCE WITH ARTHUR CASAGRANDE JULY 1964	DETAILED DECISION	SETTLEMENT		CONSTRUCT SLIGHTLY HIGHER CREST (2'-3' CAMBER) TO ALLOW FOR ANTICIPATED SETTLEMENT	FROM CASAGRANDE & BERTMAN - "BASED ON OBSERVATIONS AND EXAMINATIONS OF SEVERAL ROCK-FILL DAMS" SETTLEMENT AFTER CONSTRUCTION 2'-3' MAXIMUM MAXIMUM SETTLEMENT OCCURS DURING FIRST RESERVOIR FILLING INCREASED ϕ BETWEEN EMBANKMENT AND FOUNDATION TO 35° AFTER OBSERVING "ROUGHNESS" OF STRIPED ROCK FOUNDATION OUTER UPSTREAM & DOWNSTREAM SLOPES STEEPER THAN 1.68:1 V "NOT ACCEPTABLE", "ON THE BASIS OF JUDGEMENT OF THE MATERIALS OF CONSTRUCTION" CORE TRENCH NOT REQUIRED "IN ACCORD WITH PRESENT CRITERIA" ROCK SHOULD BE REMOVED BACK TO BASE OF		ACTUAL SETTLEMENT OF EMBANKMENT
	DETAILED DECISION	STABILITY ANALYSIS	VISUAL INSPECTION OF CLEANED ROCK FOUNDATION	ADDITIONAL STABILITY ANALYSIS TO BE PERFORMED USING $K_0=0.5$, $F_8=1.5$ WITH ϕ BETWEEN EMBANKMENT AND FOUNDATION OF 35°			
LETTER OF 18 OCT. 1964	DETAILED DECISION			SHOULD PROVIDE INSTRUMENTATION			

Table 2
Data Collection Phases

Date	Data	Purpose
- March 1961	6 Borings	Site selection report.
- October 1961	Test fills	Test suitability of rock fill materials.
- September 1962 -October 1963	Test fills	Test compaction equipment.
- October 1963	Triaxial Laboratory Tests	Preliminary material properties for fill.
- October 1963	Borings	Location of tunnels and power works.
- July 1963	160 Borings, Triaxial testing	Site conditions at main dam, borrow material for core.



PART IV -- ASSESSMENT OF STATISTICAL METHODS FOR GEOTECHNICAL ASPECTS OF DAM PROJECTS

This section summarizes the applicability of statistical methods to geotechnical engineering aspects of dam projects. The assessments derive primarily from experience gained in applying statistical techniques to Carters Project data, but also reflect work reported in the geotechnical literature. Examples are given from the Carters Project applications. While the conclusions appear to be generalizable to most dam projects, they are nevertheless based on one project. It is possible--indeed likely--that certain statistical methods which were not found to be useful on the Carters Project could be found useful elsewhere.

General Assessment of Statistical Methods

The principal conclusions of the application of statistical methods to the Carters Project data are,

1. Statistical methods do not appear well suited to the problems associated with geotechnical engineering aspects of major design decisions on new dam projects. These problems tend to involve non-quantitative uncertainties which are not amenable to mathematical analysis.
2. Statistical methods do appear well suited to the problems associated with routine aspects of data collection and management, engineering modeling, and construction control. In these problem areas, statistical methods have the potential to increase engineering efficiency and provide a traceable path between data and performance predictions.

Note should be taken that limits of the present study may bias the conclusions reached. This study addresses statistical techniques, not the broader set of methodologies often described as risk analysis or decision analysis. Specifically, methodologies such as systems risk assessment, subjective probability assessment, and multiattribute decision analysis were expressly excluded from consideration. As a result, exactly those technologies were excluded which some workers maintain to be appropriate to analyzing high-level decisions (e.g., Keeney, 1980).

Fundamental Assumptions or Hypotheses

The design and subsequent performance of any dam rest on a usually limited number of fundamental hypotheses. These hypotheses may involve aspects of site geology, the behavior of construction materials, forecast of environmental conditions, or other things. In many if not most cases of adverse performance, difficulties can be traced to the violation of one or more of these fundamental hypotheses (e.g., DeMello, 1977). This is also true of bridge and building failures (e.g., Yam, et al., 1980).

On the Carters Project one fundamental hypothesis for the design of the main embankment was an appropriate value of base friction angle to be used in performing stability analyses. This friction angle controlled the transfer of stress between the main embankment and its rock foundation, and thus influenced the overall design geometry of the embankment. In view of its influence on final embankment design, the base friction angle was an important parameter. Yet, the decision to use a base friction angle of 35° benefitted little from test data or analyses. The decision, as discussed further in Part V, was based in large measure on visual inspection of the cleaned rock surface, on analogies

with other dams, and on the intuition of experienced engineers and consultants. Technologically, it is not clear how else the decision could have been made.

Typically, the adoption of fundamental hypotheses is based more on the intuition and judgement of senior personnel than on data and quantitative analysis. Thus, statistical methods offer little guidance in dealing with fundamental hypotheses. Subjective probability, risk analysis, and other technologies which have been proposed for assessing uncertainties associated with fundamental hypotheses are outside the scope of the present report.

Aspects of Geotechnical Engineering

Unlike the fundamental hypotheses which underlie major design decisions, many of the operational aspects of geotechnical engineering for new dam projects do involve quantitative analysis. These more routine operational aspects appear to derive benefit from the use of statistical methods. Specifically, in these operational areas statistical methods appear to provide:

- (i) Increased productivity in routine data analysis and modeling.
- (ii) Increased technological capability for sensitivity studies and error analysis of engineering calculations.
- (iii) Improved sampling schemes for construction control and site characterization.

These are all areas in which statistical methods have been widely used in other fields of engineering.

The important programmatic implications to the USAE of adopting statistical methods for operational aspects of geotechnical engineering are,

- 1 Increased facility for computerization and automated data processing.
- 1 Better quality control in engineering design
- 1 More consistent quality assurance in construction

These programmatic implications are consistent with two major trends in civil engineering, (i) rapid computerization, and (ii) demand for engineering and construction quality.

Results of the present study do not support the adoption of more esoteric statistical procedures except on a special case basis or in research. Examples of these more esoteric procedures include search theory, stochastic modeling of embankment performance, and probabilistic analysis of piping potential.

On the Carters Project, the benefits gained by introducing statistical methods come primarily in routine operational areas: planning boring programs, analyzing soils test data, assessing the reliability of calculations, setting up construction inspection schemes, and so on. The benefits do not appear to come primarily in high level planning activities such as site selection and conceptual design, or in highly sophisticated modeling of engineering performance (e.g., strength, deformability, and seepage).

Carters Project Applications

Applications of specific statistical methods to data from the Carters Project were made in three general problem areas: site characterization and instrumentation, engineering analysis, and construction inspection. The intent of these applications was to exercise specific statistical methods on specific data sets to judge whether those methods appeared to offer benefits over traditional procedures. These analyses are not comprehensive nor were they intended to be so. They do not address even the majority of geotechnical engineering issues and calculations involved in the design of a major dam such as Carters. The intent was to identify likely aspects of the geotechnical engineering of new dam projects for which statistical methods

might be useful, and then to test the applicability of those methods to those aspects by performing chosen, typical analyses.

Site Characterization

Primary emphasis in this section is placed on site characterization for the reregulation dam, downstream of the main embankment. Consideration of the site characterization program for the main embankment are discussed in Part V, for the reasons stated below. The reregulation dam is a low (50') rock fill embankment with an upstream compacted impervious zone. It is founded on 20 to 50' of silty sandy alluvium overlying sedimentary rock of the Valley Physiographic Province. The embankment is approximately 2600 feet long with 3:1 slopes.

The reregulation dam rather than the main embankment was chosen the subject of primary work in this section for three reasons:

- i. The reregulation dam rests on a soil foundation rather than on rock as the main embankment does.
- ii. Extensive site investigation, soil testing, and analysis was conducted to establish engineering properties at the reregulation dam, whereas little testing or analysis of site investigation data was conducted at the main dam, owing to its high quality rock foundation.
- iii. Results of the site characterization program at the reregulation dam provided quantitative input to the engineering analysis of the reregulation dam, whereas, at the main embankment engineering analyses focused principally on the performance of embankment materials.

Analysis of the Boring Program

The boring program at the reregulation dam consisted of approximately 63 standard penetration test borings located along the axis of the dam and along a section perpendicular to the axis at the spillway location (Figure 14). Additional borings were concentrated in a zone along the axis between stations 27+50 and 37+00 where deep solution activity was discovered in the limestone foundation.

An interpreted profile along the axis is shown in Figure. 15. The soil conditions along the axis can be roughly divided into three sections on the basis of SPT blow counts: Station 4+00 to 16+50, 17+00 to 24+50, and 25+00 to 32+00. Blow count data for the three sections are shown in Figs. 16, 17, and 18; and summarized in Table 3.

Table 3
SPT SUMMARY STATISTICS FOR REREGULATION DAM FOUNDATION

section	mean (bpf)	standard deviation (bpf)	coefficient of variation	length (feet)
Station 4+00 to 16+50 elevation 654-676	4.8	2.9	0.60	1250
Station 17+00 to 24+50 elevation 652-671	6.9	2.8	0.41	750
Station 25+00 to 32+00 elevation 650-670	8.9	4.4	0.49	700

The average blow counts increase from right to left across the dam axis. The soils underlying the right wing of the dam have very low N values.

The blow count data in each section more or less follow a Normal distribution of relative frequency. This trend is seen in the probability-grid

plots, on which Normally distributed data plot as straight lines. The envelopes shown on the plots are Kolmogorov-Smirnov bounds (at 5% and 1%, respectively, for the narrower and wider envelopes). Deviations outside of these envelopes indicate that the data do not exactly fit a Normal curve in a statistical hypothesis testing sense. The coefficients of variation in each section are approximately constant at 50%. As per our understanding of USAE design procedures for static loading, the blow count data were not corrected for overburden. The spatial structure of the blow count data was investigated both directly on the raw blow counts and on detrended blow counts for which a regression line with depth was removed.

Results

The autocovariance function for the non-detrended average blow counts in each boring is shown in Figure 19, estimated using the moment estimator technique,

$$C_N(r) = E[(N_i - m_N)(N_{i+r} - m_N)] \quad (11)$$

in which $C_N(r)$ is the autocovariance of N as a function of separation distance r , N_i is the blow count at location i , N_{i+r} is the blow count at location $i+r$ and m_N is the average blow count. The value of $C_N(r)$ at $r=0$ is the variance of the boring-averaged blow count data across space. From Figure 19 the variance of average blow counts in each boring is about 6.0 bpf^2 , and the autocorrelation distance is about 350'.

A corresponding autocovariance function for individual data at elevation 660 is shown in Figure 20. The variance of these data is 12.5 bpf^2 , and the autocorrelation distance, r_0 , is about 200 to 250'.

The strong spatial structure in both cases is due to the trend of blowcounts along the embankment axis. One expects a zone of roughly $2r_0$ to have statistically similar properties, in this case about 600 to 700'. This is on the same order as the three sections separated in Figures 16, 17, and 18. When each of the three sets of data is analyzed separately, a different mean blow count is found for each. This reduces spatial correlation because it in effect subtracts a spatial trend out of the data. Autocorrelation reflects the spatial pattern in a set of data which is unaccounted for by lumping all the data into one group. Thus, when any spatial trend is removed, the autocorrelation is reduced. When the three sets of data are analyzed separately, the autocorrelation distance in each becomes smaller than the typical separation distance between adjacent borings. Thus, conclusions become difficult to draw.

Comparison of Estimation Techniques

Three techniques were used to estimate autocovariance functions in the analysis of site characterization data, the moment estimator, BLUE minimization estimator, and maximum likelihood estimator. These led to different results.

The moment estimator uses the autocovariance function calculated directly from the observed measurements as an estimator of the autocovariance of the underlying spatial process:

$$C_z(r) = \left(\frac{1}{n-1}\right) \sum [(z_i - m_z)(z_{i+r} - m_z)] \quad (12)$$

in which n = the number of data pairs at separation distance r .

The BLUE minimization estimator uses the autocorrelation function that minimizes the squared error between estimated and observed soil properties at the measurement points as an estimate of the autocovariance of the underlying spatial process. That is, soil properties are estimated at each of the observed points by removing that measurement from the data base and using the remaining data to estimate it via Equation 19. That autocovariance function which minimizes the variance of Equation 20 is taken as the estimate. This is a parametric model (i.e., the shape of the autocovariance function is specified).

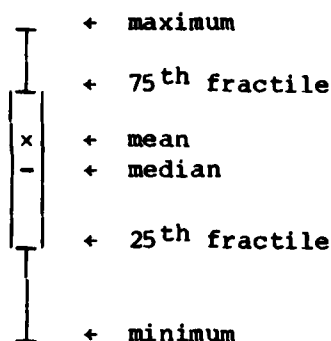
The maximum likelihood estimator uses the autocorrelation function that maximizes the conditional probability of the measurements actually made (i.e., the 'likelihood') as the estimator of the autocovariance of the underlying spatial process:

$$\min_{C_z(r)} L[z_1, \dots, z_n] = \min_{C_z(r)} MN(\underline{\beta}\underline{x}, \underline{\Sigma}_z) \quad (13)$$

in which $L[\underline{z}]$ = the likelihood or conditional probability of the vector of data \underline{z} , $MN()$ = the multivariate normal probability density function, $\underline{\beta}$ is a vector of regression coefficients for the mean trend of the data, \underline{x} = the matrix of location coefficients each row of which is $\langle 1, x_1, x_1^2, x_1^3, \dots, x_1^k \rangle$ where k is the order of the regression surface, and $\underline{\Sigma}_z$ = the covariance matrix of the observations calculated via the autocovariance function. This is a parametric model (i.e., the shape of the autocovariance function is specified), but it is asymptotically unbiased and minimum variance.

Results of applying the three techniques above to the blow count data at the reregulation dam site are shown in Figures 21, 22, and 23. The upper plot

in Figure 21 shows the mean estimates calculated as per Equation 9. The lower plot shows variation about the mean estimate, represented as:



The plot in Figure 22 shows the autocovariance function estimated by minimizing the error of the weighted sum interpolation scheme of Equations 19 and 21. The plot in Figure 23 shows the autocovariance function estimated by minimizing Equation 13. In both Figures 22 and 23 a simple exponential equation was used to model autocovariance, such that $C_z(r)$ was set equal to $V_z \exp(-r/r_0)$, in which V_z is the variance of z , r is distance, and r_0 is a constant (i.e., the autocovariance or autocorrelation distance). For horizontal autocovariance, typical of many of the results the three methods led to are the following:

method	autocovariance distance	measurement noise
moment estimator	350 feet	0
BLUE minimization	150	0
maximum likelihood	200	0

From a statistical point of view, the maximum likelihood method is generally thought to have preferable properties to the moment estimator. The statistical properties of the BLUE procedure are not well studied. The

comparison of estimation techniques for autocovariance is important since the autocovariance function is used to indirectly estimate measurement noise. Interestingly, while the three methods in this case give clearly different estimates of autocovariance distance, each gives the same estimate of measurement noise. This may or may not be fortuitous.

Measurement Noise

Measurement noise in in situ data can be estimated using the autocovariance function. Measurements are represented by the common statistical model,

$$z = x + e \quad (14)$$

in which z =the measured value of some property (here blow count), x =the real or "true" value, and e =measurement error. The autocovariance function of z is

$$C_z(r) = E[(z_i - m_z)(z_{i+r} - m_z)] \quad (15)$$

and correspondingly for x and e . After algebraically expanding and rearranging the right hand side (RHS) of Equation 15, the autocovariance of z can be shown to relate to the autocovariance of x and e as,

$$C_z(r) = C_x(r) + C_e(r) \quad (16)$$

Making the assumption that measurement noise is independent from one test to another,

$$C_e(r) = \begin{cases} V(e) & \text{at } r=0 \\ 0 & \text{elsewhere} \end{cases} \quad (17)$$

That is, the autocovariance of e is a spike at $r=0$. Thus, the autocovariance of z should be a smoothly trending function at $r>0$ and have a spike of height $V(e)$ added to it at $r=0$ (Figure 24). By extrapolating the empirical autocovariance of z back to the origin at $r=0$ an estimate of both $V(x)$ and $V(e)$ can be made. For many geotechnical tests in situ measurement noise can account for 30 to 50% or more of the observed data scatter.

An interesting finding from the blow count data at the reregulation dam is that the measurement error in the data appears to be very small or negligible. This is surprising. SPT blow counts are widely thought to be noisy measurements and have been shown elsewhere to have quantitative noise components in excess of 50%. This lack of measurement noise in the reregulation dam data can be inferred from the autocovariance function. There is essentially no discernable spike at $r=0$, and thus the variance of the measurement error $V(e)$ would appear to be about zero.

A primary source of measurement noise in SPT data is blow by blow variation in energy delivered to the drill rod. The effect of this variability on the noise in (i.e., variability of) N is related to the derivative of N with respect to energy ratio ER as,

$$V(N) = (dN/dER)^2 V(ER) \quad (18)$$

Figure 25 from Campanella, et al. (1984) suggests that the variability delivered energy decreases as m_N decreases. That is, for small N the variance $V(ER)$ is small or at least modest. Figure 26 from McLean, et al. (1975) suggests that the sensitivity of N to ER (i.e., the derivative) is also small for small m_N . Therefore, both terms of the LHS of Equation

18 are small or modest, and thus the noise introduced into N should be small as well. This is an after the fact explanation, but is consistent with the observation that noise in the reregulation dam SPT data is small.

Weighted Estimation and Interpolation of Soil Properties

It has been suggested in the literature that the statistical structure of in situ measurements can be used to obtain better estimates or interpolation of soil properties than can be obtained from averages or trends alone. At the simplest level this is accomplished by using a weighted sum of the observed data to estimate properties at unobserved locations, in which the weights are non-uniform and optimized to take account of spatial correlation. In the civil engineering literature this is usually called a best linear unbiased estimate (BLUE). In mining an equivalent technique goes under the name kriging. The BLUE technique was applied to the reregulation blow count data to test its practicality and value.

Barring other information, the best estimate of soil properties at a point is found by taking the mean or mean trend to all the data. However, noting that soil properties near by the point to be estimated may be more similar to the properties at that point than are properties at far distant locations, one might form a better estimator than the overall mean by giving preferential importance to those close by measurements. The easiest such estimator is the weighted sum,

$$z_0 = \sum w_i z_i \quad (19)$$

in which z_i = the i^{th} of n measurements and w_i = the weight applied to the i^{th}

measurement such that $\sum w_i = 1$. Clearly, the mean has this same form in which each of the $w_i = 1/n$.

The variance of z_0 can be derived to be,

$$V(z_0) = \sum \sum w_i w_j C_{z_i z_j} \quad (20)$$

in which $C_{z_i z_j}$ is the covariance of the i^{th} and j^{th} measurements. Expanding the left hand side (LHS) and minimizing $V(z_0)$ subject to the constraint $\sum w_i = 1$ leads to the optimal weights \underline{w}^* ,

$$\left(\frac{\underline{w}^*}{\lambda} \right) = \begin{bmatrix} \underline{C_z} & \underline{1} \\ \underline{1}^T & 0 \end{bmatrix}^{-1} \begin{bmatrix} \underline{C_0} \\ 1 \end{bmatrix} \quad (21)$$

in which λ = a Lagrange multiplier, and $\underline{C_z}$ = the covariance matrix of the measurements \underline{z} , $\underline{C_0}$ = the vector of covariances of the point to be estimated with the locations where measurements were made, and $\underline{1}$ is a vector of one's. $\underline{C_z}$ is found from the autocovariance function of the z_i . For the case in which all the measurements are independent (i.e., widely spaced), $\underline{C_z} = \underline{I} V(z)$ and $\underline{w}^* = \{1/n, \dots, 1/n\}$. z_0 reduces to the spatial mean.

Equation 21 was applied to the boring-averaged blow count data for the reregulation dam to test the effectiveness of statistical mapping. In turn, each of the data was removed from the data set and the remaining $(n-1)$ data were used via Equations 19 and 21 to estimate the removed point. $\underline{C_z}$ and $\underline{C_0}$ were evaluated from the autocovariance function. Then the estimated value of z was compared to the actually measured value. This result is shown as a histogram of normalized values in Figure 27. The normalized values are,

$$E = (1/n) \sum \left(\frac{z_i - z_0}{s(z_0)} \right) \quad (22)$$

in which $s(z_0)$ is the predicted standard deviation of z_0 from Equation 20. In theory the error ratio data should have mean zero and unit standard deviation and approach a Normal distribution function as $n \rightarrow \infty$. The results are close to this theoretical distribution. The variance of the data as a whole is 6 bpf². The variance of the z_0 is 4 bpf². Thus, a 33% increase in precision is obtained by using the BLUE estimator over using the spatial mean. Such an improvement is modest but potentially useful, especially as statistically based computer-aided design systems come into more widespread use. Krigel interpolations, which are similar to BLUE interpolation, are in common use in the mining industry for ore reserve estimation.

An analysis similar to that conducted on blow count data was conducted on depth to rock in the individual borings. The intent of such analyses is to determine whether soil volume (e.g., borrow) estimates might be made more precisely. The results of the depth-to-rock analyses were similar to those summarized above for blow count. Figure 27 shows the autocovariance function for depth-to-rock across the reregulation dam site. Distance is in feet. The autocorrelation distance of depth-to-rock is approximately 200 feet. The end result of these calculations was similar to that for the blow count data. A point-to-point decrease in estimation variance of up to 30% could be obtained through stochastic interpolation as compared with nearest neighbor techniques. This result is consistent with findings in the mining literature (e.g., Journel and Huijbregts, 1978).

Statistical Soil Engineering Profiles

Returning to the measurement model of Equation 14, but now expressing x in terms of the measurements z , and applying a measurement bias correction (i.e., calibration) factor B ,

$$z = Bx + e \quad (23)$$

The variance of x as a function of the variances of z and e can be found--from methods discussed later in this part of the report (Equation 32)--to be,

$$V_x = (1/m_B^2)(V_z - V_e) + m_z^2 \Omega_B^2 \quad (24)$$

in which Ω_B is the coefficient of variation of the uncertainty in the appropriate value of the bias correction B . For example, for field vane (FV) data, Ω_B is found from the scatter of calibration data of the type compiled by Bjerrum (1962).

Although measurement noise can be estimated and removed from the data scatter, it still retains an effect on statistical uncertainty

$$V_{mz} = V_z/n \quad . \quad (25)$$

The overall error in the estimate of soil properties at any one point is found by combining the individual contributions of soil variability, measurement bias and statistical error, to obtain,

$$\begin{aligned} \left(\begin{array}{c} \text{uncertainty} \\ \text{in } x \end{array} \right) &= \left(\begin{array}{c} \text{soil} \\ \text{variability} \end{array} \right) + \left(\begin{array}{c} \text{statistical} \\ \text{uncertainty} \end{array} \right) + \left(\begin{array}{c} \text{bias} \\ \text{uncertainty} \end{array} \right) \\ V_x &= (1/m_B^2)(V_z - V_e) + V_z/n + m_z^2 \Omega_B^2 \end{aligned} \quad (26)$$

Since the contribution of random measurement error appears only in its effect

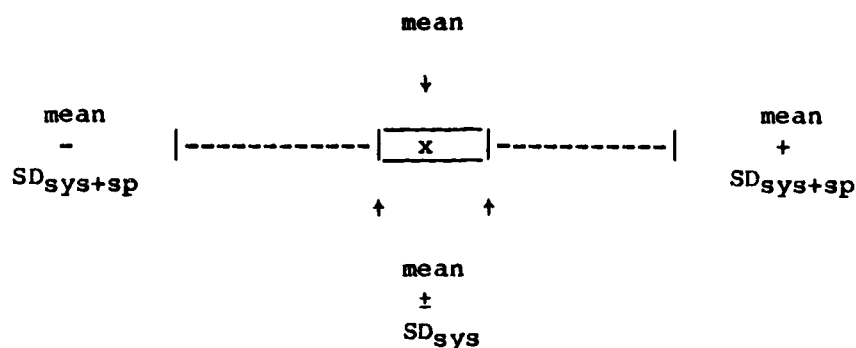
on statistical error, this means that V_x in specific instances can be considerably less than the data scatter variance V_z .

In separating spatial variability and systematic error, it is easiest to think of spatial variation as scatter about the trend and to think of systematic error as uncertainty on the trend itself. The first envelope is due only to soil variability after random measurement error is removed. The second envelope is due to statistical error and measurement bias.

Because uncertainty comprises both spatial and systematic components, the magnitude of uncertainty which must be dealt with depends on the volume of soil mobilized in the limiting state. For limiting states that depend on average properties and large volumes of soil, the spatial component of variability averages out. The size effect factor $R < 1.0$. For small volumes this component contributes in its full magnitude to uncertainty, $R = 1.0$. On the other hand, for limiting states that depend on extreme elements the spatial component of variability increases in importance as the volume of soil mobilized becomes larger. The size effect factor $R > 1.0$. Typically, R augments the first term of the RHS of Eqn. 26, becoming as a final result

$$V_x = R (1/m_B^2) \{V_z - V_e\} + V_z/n + m_z^2 \Omega_B^2 \quad (27)$$

The results of the above analysis on the blow count data under the reregulation dam are shown in Figures 28a,b, and c, in which the mean profile and standard deviation envelopes are denoted as:



in which SD_{sys} is the systematic standard deviation and SD_{sys+sp} is the systematic plus spatial standard deviation. Also shown in the figure is uncertainty in ground water level, denoted by a mean and $\pm SD_{spatial}$. The implications of this "statistical profile" on engineering analysis of the reregulation dam are discussed below.

Applications of Statistical Pattern Recognition to Soil Profile Data

An area of statistical technology currently gaining interest is the application of pattern recognition, scene analysis, and other aspects of artificial intelligence (AI) to data analysis. Related applications of knowledge-based expert systems to geological data processing is already in development (Part II).

An attempt was made to apply such techniques to site characterization data for the reregulation dam. Specifically, statistical methods of pattern recognition were applied to identifying stratification in the foundation soils simply from SPT data, without the benefit of a human interpreting the data. The purpose of such methodology is to automate a larger fraction of routine data analysis, thereby increasing engineering productivity.

Fig. 29a shows the boring lay out at the site of the reregulation dam and a set of six borings used as a test data base for applying statistical pattern

recognition to the problem of identifying stratification. These borings were chosen because they are roughly aligned and parallel with the presumed direction of deposition of the alluvial sediments. The intent in selecting these particular borings was to reduce heterogeneity as much as possible among the soils encountered in adjacent borings.

The visual classifications of the soils according to USCS classes in these borings are shown in Fig. 29b. In a simplified way, each of the logs shows a brown silty sand (SM), overlying inorganic silt (ML), in turn overlying brown silty sand (SM).

A two stage procedure was used in processing the data. In the first stage a version of regional merging was used to group data into classes. The entire set of data is grouped into a matrix. The columns of the matrix are data from individual borings, the rows are data at constant elevations. For each datum, all adjacent data (i.e., up, down, left right, diagonally) in the matrix are tested for similarity. When 'similar' data are found they are merged into a zone (i.e., a stratum). The process is repeated to form larger and larger zones, until no further 'similar' neighbors are found.

In the second stage an interactive form of regression analysis is performed to identify linear trends with depth in the data from each boring, starting with the zonation identified by regional merging. The dividing points between strata are refined throughout the interaction.

Results of the pattern recognition procedure are shown in Fig. 29c, compared with stratification of Fig. 29b based on visual classification. In fact, the agreement is fairly good, and suggests that advanced statistical programming techniques such as AI may offer the opportunity of increasing the automated data processing of soils data.

Engineering Analysis

Uncertainty Analysis of Engineering Calculations: Methodology

Engineering analysis uses soil property estimates made from measurements in the laboratory and measurements in the field by incorporating them in models based on engineering mechanics (Figure 30). These models relate soil properties and other aspects of a design to predicted performance. In traditional design, single value estimates of properties are entered into the model and a single value estimate of performance is calculated. To test the sensitivity of a prediction to uncertainty in soil properties, a number of calculations are made and the resulting predictions plotted as a function of the input parameters. Sensitivity analyses of this type are more difficult when more than one parameter is uncertain or when uncertainties are not independent from one parameter to another.

Using statistical methods, calculations can be based on more than single value estimates, thereby somewhat reducing conservatism. In its simplest and currently most useful form, uncertainty is propagated through an analysis in the form of standard deviations (or variances) and correlation coefficients (or covariances). The standard deviations and correlation coefficients are translated to corresponding standard deviations and correlation coefficients on predictions. For example, the result of a stability calculation would be a best estimate of factor of safety (FS) and a corresponding standard deviation of FS.

UNCERTAINTY PROPAGATION

Operationally, uncertainty is propagated through an analysis using

first-order (i.e., linear) approximations. For a model relating an input parameter x to a prediction y through the model $g(x)$,

$$y = g(x) \quad . \quad (28)$$

A Taylor's series expansion truncated to linear terms yields the approximations,

$$m_y \hat{=} g(m_x) \quad , \quad (29)$$

$$s_y^2 \hat{=} \left(\frac{dg}{dx}\right)^2 s_x^2 \quad (30)$$

in which $\hat{=}$ indicates first-order (i.e., linear) approximation. In words, the mean or best estimate of the prediction y is the function of the mean or best estimate of the parameter x ; the variance (squared standard deviation) of the prediction y is the product of the variance of the parameter x and the square of the derivative of y with respect to x . The derivative of y with respect to x might be thought of as the sensitivity of y to changes in x . The result is based on a linear approximation to $g(x)$, but for most geotechnical problems it is sufficiently accurate. In those cases for which a linear approximation is not sufficiently accurate, other mathematical techniques for calculating the uncertainty in performance predictions are available. These are described in the report "Error analysis for geotechnical engineering," (Contract Report GL-87_) and briefly introduced below.

If the prediction y depends on a set (i.e., vector) of parameters, the equivalent forms are,

$$m_y \hat{=} g(m_{x1}, \dots, m_{xn}) \quad , \quad (31)$$

$$s_y^2 = \sum_i \sum_j \frac{dg}{dx_i} \frac{dg}{dx_j} C_{x_i, x_j} \quad (32)$$

Two special cases deserve note because they are common in practice and have simple results. For the case in which y is a linear combination of a set of independent parameters $y = \sum a_i x_i$ the variance of y is exactly,

$$s_y^2 = \sum_i a_i^2 V_{x_i} \quad (33)$$

For the case in which y is a power function of a set of independent parameters, $y = \pi x_i^{a_i}$, the variance of y is approximately,

$$1 - \Omega_y^2 = \sum_i (1 - a_i^2 \Omega_{x_i}^2) \quad (34)$$

which for small coefficients of variation (<30%) reduces to,

$$\Omega_y^2 = \sum_i a_i^2 \Omega_{x_i}^2 \quad (35)$$

SIZE OR VOLUME OF MATERIAL EFFECT FACTOR

The importance of spatial variability on calculated predictions depends both on the mode of performance and the volume of soil influenced. For modes of performance which depend on average soil properties within a large volume of soil, such as consolidation settlement, spatial variability partially averages out. These are called averaging modes. For modes which depend on worst conditions, for example sliding along a (small volume) discontinuity, spatial variability is accentuated. These are called amplifying modes. In either

case, a size effect multiplier R is used to account for the effect of soil volume on the importance of spatial variability.

The multiplier R reflects size effect by reducing or increasing the spatial variance of a soil parameter as that variance is propagated through a calculation. For averaging modes, R is less than 1.0. For amplifying modes, R tends to be greater than 1.0, but there is usually also an accompanying change in the mean value.

Consider the uncertainty in one-dimensional consolidation of a uniform clay. For normally consolidated clay final consolidation settlement can be calculated by the formula,

$$\rho = \sum_{i=1}^n H_i m_v \Delta\sigma_v' \quad (36)$$

in which ρ =settlement, H_i =thickness of the i^{th} of n layers, m_v =coefficient of volume change, and $\Delta\sigma_v'$ =change in vertical effective stress. Given spatial variation of m_v reflected in a mean m_{mv} and a variance $V_{mv}=s_{mv}^2$, the mean settlement is,

$$m_\rho = \sum_{i=1}^n H_i m_{mv} \Delta\sigma_v' \quad , \quad (37)$$

in which $\Delta\sigma_v'$ is assumed known.

The total variance of ρ , however, cannot be calculated simply by propagating s_{mv}^2 through Equation 36, because s_{mv}^2 has both spatial and systematic components:

$$s_{mv}^2 = s_{mv}^2(\text{spatial}) + s_{mv}^2(\text{systematic}) \quad (38)$$

The systematic component of variance does propagate through the equation directly because systematic error affects each soil layer the same way. Spatial variability, on the other hand, is realized differently in each layer, and thus higher than average m_v in one layer will balance lower than average m_v in another.

Considering only spatial variances, if each of n soil layers were independent of all others, and if the spatial variance of average m_v among layers were V_{m_v} , then the variance in settlement would be,

$$s_{\rho}^2 = \sum_{i=1}^n (H_i \Delta \sigma_v')^2 s_{mv}^2 \quad (39)$$

In this special case, as the number of layers increases--with $\sum H_i$ constant--mean settlement remains constant but the standard deviation of settlement goes down. The coefficient of variation for ρ would be

$$\Omega_{\rho} = \frac{\{ \sum (H_i \Delta \sigma)^2 \}^{1/2} s_{mv}}{\{ \sum H_i \Delta \sigma \} m_{mv}} \quad (40)$$

$$= \frac{\Omega_{mv}}{\sqrt{n}} \quad (41)$$

In words, the spatial variations from one layer to another average as n increases. Therefore, if one arbitrarily divided an homogeneous clay into a number of hypothetical layers, and if the variance of m_v were then applied to each as if the layers were independent, then the calculated variance of

settlement could be made as small as desired simply by increasing the number of hypothetical layers.

Clearly, this artifact has nothing to do with the soil or the true uncertainty about settlement. It comes from the inappropriate assumption that soil properties in the respective layers are mutually independent. They are not. The autocorrelation of soil properties in space implies that as the layers are made thinner, the soil properties in adjacent layers become more correlated. Soil elements near one another tend to have similar properties and soil elements widely separated tend to have no association.

Mathematically, the mean and variance of an average or sum, such as Eqn. 36, can be found from the autocovariance function, and the results of such calculations can be tabulated or graphed as a size effect multiplier R . Results for the special 1D case with an exponential autocorrelation function are shown in Fig. 31. For routine use in calculations, R is multiplied by the spatial contribution to variance.

RELIABILITY INDEX

Factor of safety, F , is generally taken to be the ratio of capacity to demand. Typically, F is expressed as a single estimate based on conservative estimates of capacity and generous estimates of demand. With statistical methods the conservative F is replaced by a factor of safety based on best estimates. This so-called central F is the ratio of mean capacity to mean demand, and is the best estimate or mean F . The uncertainty in F , rather than being implicitly incorporated in a single estimate, is stated as a variance or standard deviation. Thus, two measures result, a mean m_F and a variance V_F .

In assessing reliability both the mean F and standard deviation play a role. Reliability is related to the probability that the F realized in service

exceeds 1.0. The mean F alone is insufficient to judge this reliability, as can be seen in Figure 32. From the results of two calculations, the mean F of calculation number two is greater than the mean F of calculation number one, but because the standard deviation of number two is also greater than that of number one there is a larger probability of F_2 being less than 1.0 than of F_1 being less than 1.0. These probabilities are simply the areas under the respective probability distributions within the interval $0 < F < 1.0$. For convenience, m_F and s_F are combined in the reliability index

$$\beta = \frac{m_F - 1.0}{s_F} \quad (42)$$

This index measures the number of standard deviations separating the best estimate of F from its nominal failure value (1.0). For two parameter distributional shapes β is monotonically related to the probability that $F < 1.0$.

OTHER METHODS FOR UNCERTAINTY ANALYSIS

The approach to propagating uncertainty through an engineering model used here is based on a first-order or linear propagation of variance. This is a common technique and is called by different names in the many disciplines to which it finds application. It is sometimes called "first-order second-moment" (FOSM) analysis, and sometimes simply "error analysis." However, there are other ways to analyze the effect of input uncertainties on output uncertainties. These can be particularly useful for dealing with highly non-linear calculations. Among the more often encountered of these other methods

in civil engineering practice are adjoint methods, simulation, and response surface techniques.

Adjoint techniques evaluate the proportionate effect of a perturbation in input parameters on the resulting perturbation in an output prediction. That is, they lead to an evaluation of the quantity $\{(\Delta y_j / \Delta x_i) x_i / y_j\}$, in which y_j is the j th component of the prediction and x_i is the i th input parameter. Adjoint techniques are conveniently applied to large numerical models involving the solution of systems of linear equations. By manipulating the linear algebra of such solutions, adjoint results can be obtained in the course of computations. While adjoint techniques are usually used to obtain sensitivities of a model rather than to perform quantitative uncertainty analysis, the results can be used to numerically obtain derivatives, and thus to provide the means for first-order variance propagation.

Monte Carlo simulation uses many repetitions of deterministic calculations in which values of input parameters are randomly generated from specified probability distributions. The result of simulation is a set of many predictions of each output parameter which are treated as empirical data from which statistical inferences of the means, variances, etc. of output predictions can be made. An advantage of simulation is simplicity. It requires none of the mathematics of variance propagation, adjoint analysis, and related techniques. On the other hand, simulation has three important limitations. It is expensive because the deterministic model must be run many times. For example, at least several hundred trials are typically needed. It requires not only means and variances of input parameters, but entire probability distributions. These may be ambiguous or arbitrary. Finally, the

components of uncertainty are lumped together in simulations. Thus, differing effects are hard to unravel. Nevertheless, simulation is an important tool when a model is complicated, nonlinear, involves logical branching, or on other occasions when variance propagation and related techniques cannot be used.

Response surface techniques are related both to variance propagation and simulation, finding their most frequent use with models that are numerical, possibly implicit, difficult to analytically propagate variance through, and expensive to run. Response surface techniques are closely related to regression analysis. Multiple runs of the model are made in the vicinity of the mean of the input parameter values and a regression surface of chosen complexity is fit to the output predictions obtained. Thus, the function $y(x)$ is approximated by a nonplanar surface rather than a plane as in first-order analysis. This regression surface is presumably less complicated than the model function itself, and yet can still be used as an approximation on which variance propagation or other techniques can be used. At the same time, many fewer runs of the model are made than with simulation, and thus cost is reduced. Response surface approaches are often applied to risk analysis problems associated with nuclear power and waste facilities, and to structural reliability problems.

Stability Calculations of Reregulation Dam

The results of statistical analysis of the site characterization data at the reregulation dam are discussed in the previous section. These data are primarily taken from standard penetration test borings in the vicinity of the reregulation dam. Analyzed 'design profiles' of blow count and depth-to-rock were presented as Figures 29a,b,c. In addition to the field data, a

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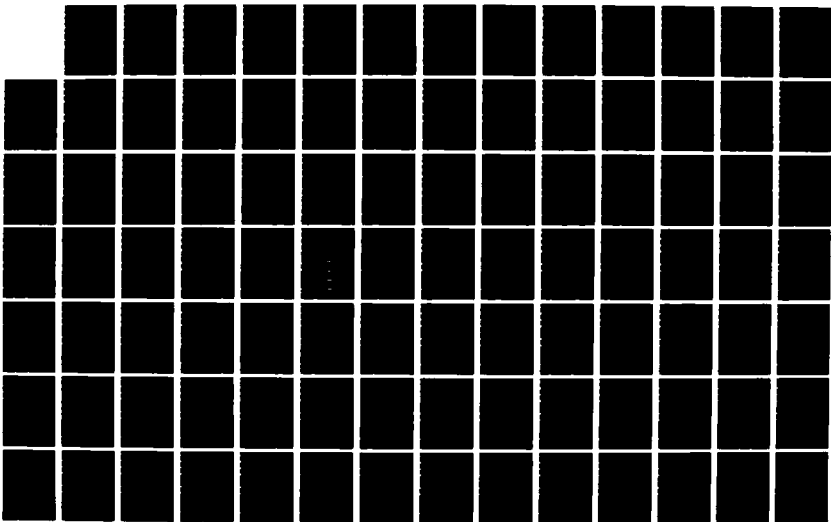
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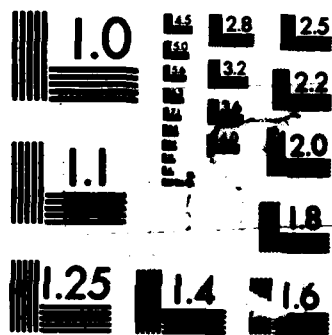
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substantial number of laboratory measurements were made. The numbers and types of these are shown in Table 4 and Figures 33a,b,c and 34a,b,c. A typical Mohr diagram of undrained strength properties of borrow materials from areas 1 and 2 are shown in Figure 35. These laboratory values were used for estimating strength parameters in the embankment.

RELIABILITY OF STABILITY CALCULATIONS FOR THE REREGULATION DAM

An uncertainty analysis of embankment stability based on the techniques presented above was performed on the reregulation dam cross-sections at Station 6+50, 10+50, 26+00, and 29+50. A typical result is shown in Table 5. This analysis was based on standard USAE analysis procedure using a method of slices approach with a circular arc failure surface as per manual EM-1110-2-1902 (1APR70). Calculation results on file at the Mobile District were used as a starting point for the analysis.

The intent of statistical uncertainty analysis of stability calculations is to determine the importance of uncertainties or errors in geotechnical parameters on the reliability of calculated predictions of engineering performance. In the case of embankment stability, the intent is to assess how site characterization uncertainties affect the reliability with which a factor of safety can be calculated. The reliability so calculated using statistical techniques is not the reliability of the embankment per se, but rather the reliability with which the prediction of embankment performance can be made. That is, the uncertainty analysis indicates the uncertainty in the analysis, not the embankment.

To accomplish the uncertainty analysis three steps were taken:

1. A 'design profile' comprising a mean profile of soil strength parameters with depth and standard deviation envelopes on those parameters was inferred statistically from the site characterization data. The mean profile is an expression of the best estimate of strength with depth. The standard deviation envelopes are of two types, one summarizing uncertainty in soil properties from point to point in the foundation or embankment, the other summarizing uncertainty in the 'best estimate' or mean strength.
2. The design profile of mean and standard deviations was propagated through the stability calculation based on method of slices to obtain a best estimate or mean factor of safety against slope instability and a corresponding standard deviation of factor of safety. This standard deviation is calculated from two components, one reflecting spatial variation and the other reflecting systematic error.
3. The mean and standard deviation(s) of factor of safety were combined in a reliability index for the stability calculation,

$$\beta = \frac{m_F - 1.0}{s_F} \quad (43)$$

In the results shown in Table 5, the mean F against instability on the most critical deterministic failure surface is 2.07, using the best estimates (i.e., means) of soil strength taken from laboratory data. This calculation

is the same as the deterministic calculation. For this particular case, the critical circle lies entirely within the embankment, and does not mobilize foundation soils. The standard deviation of F is $s_F = 0.11$, giving a reliability index of $\beta = 10$.

To place this result in context, the nominal probability of $F < 1$ corresponding to β is shown in Fig. 36. This probability is found by assuming the uncertainty in F to have a Normal or bell-shaped distribution. This corresponds to the probability that even though the calculated best estimated of F is greater than 1.0, if the 'correct' parameter values been used one would have found $F < 1$. For the present example, the frequency distribution of soil properties is not badly approximated by a Normal curve, and thus the Normality assumption is perhaps not inappropriate. However, in the general case there is often little reason to select a Normal shape for the frequency curve over many other possibilities and therefore the probability calculated on the basis of this assumption is said to be 'nominal.' It is used for primarily for comparison with other design cases.

The critical failure circle in Fig. 37 lies entirely within the compacted fill of the embankment. Because this material is controlled during compaction, its engineering properties are less variable than those within the natural foundation soils underneath the embankment. Calculations of limiting equilibrium stability at different sections where the critical circles pass through significant sections of natural soils have both different mean factors of safety and different standard deviations. The reliability index was used to evaluate consistent factors of safety for the two cases.

Fig. 38 shows the relation between mean factor of safety, coefficient of variation of factor of safety, and reliability index. For the failure surface

and design geometry of Fig. 37 the coefficient of variation of F is about 0.05. For critical circles through the natural foundation soils the coefficient of variation is typically more like 0.2. To achieve a reliability index of, say, $B=4$ in the first case only requires a mean (i.e., target) factor of safety of 1.2. To achieve $B=4$ in the second case requires a mean factor of safety of 1.8. Uncertainty in an engineering prediction influences the reliability of a best estimate. To achieve the same reliability for two design conditions usually requires different values of mean F .

Wedge Analysis of Main Embankment

Uncertainty analyses were also performed on the wedge-type failure analysis upon which the stability of the main embankment was evaluated. A large number of wedge analyses were performed during the design stage of the Carters Project, investigating different wedge geometries, numbers of wedges, rock and soil fill properties and so on.

The critical wedge geometries for stability are shown in Figs. 39 and 40. These are triangular wedges with base along the embankment-foundation interface, and backside vertically through the impervious core. Both analyses assumed end of construction condition. The upstream analysis assumes no water behind the dam. The downstream analysis assumes full pool and hydrostatic water pressures along the vertical face.

Uncertainty analysis for this wedge failure mode is straightforward. There are few input parameters to the calculation, and the failure surface geometry is specified as the least favorable to stability. The input parameters are:

Estimated: γ_3 = average weight density of the zone 3 fill.
 γ_2 = average weight density of the zone 2 fill.
 ϕ_3 = base friction angle for zone 3.
 ϕ_2 = base friction angle for zone 2.
 K_0 = lateral earth pressure in zone 1.

Given: γ_w = weight density of water
 h_w = water height behind the wedge
 h = fill height behind the wedge

Based on results of the test fill experiments and a limited number of density tests on the as placed rock fill, some information is available on the variability of dry density in the rock fill. Estimates of the variance of γ for two rock fill volumes can be obtained from the construction records, these are for the test volumes of construction density tests and for the volume of test fill experiments. To extrapolate to larger fill volumes, the autocorrelation function of dry density is presumed to have an exponential shape. This assumption appears good for core material, but its use for the rock fill is simply by analogy. By fitting a line to the two known points a rough first-approximation is obtained for the variability of larger volumes of fill. This extrapolation implies that the uncertainty in the average dry density of the rock fill, measured as a coefficient of variation, is about,

$$\Omega_{\gamma d} \approx \sqrt{V_{\gamma d}/m_{\gamma d}} \approx 0.01 \quad (44)$$

The uncertainty in the average dry density of the impervious case is found directly from the compaction control statistics. From Fig. 45 the coefficient of variation of the as-placed dry density (sand cone) measurements in the impervious zone is about 3%. The correlation distance in number of control tests is approximately 20 tests (Fig. 60). On average there are about 5 tests per 10,000 cubic yards of materials. Thus, the autocorrelation distance expressed in fill volume is about 40,000 cubic yards. Overall, the

volume of the impervious zone is about 10^7 cu. yd. Therefore, as a first-approximation, the size effect factor R for average dry density in the entire impervious zone is about

$$R = (V/2v_0)^{-1} = (10^7/2.4 \times 10^4)^{-1} = 0.01 \quad (45)$$

Thus, the coefficient of variation in average γ_d due to variability of the placement is only about 0.3% (i.e., $\sqrt{R} \Omega = 0.1(3\%)$). This uncertainty is small primarily because the compaction is controlled for uniformity.

A typical error analysis for the wedge stability computation is shown in Table 6. The reliability index against the Corps criterion of minimum FS = 1.5 is $\beta=2.4$. Against the failure criterion of FS = 1.0 the reliability index is $\beta=10$. As for the reregulation dam, the reliability of the stability calculation is high. The principal question about the calculation of stability for the main embankment, however, is not that the estimated uncertainty is larger than acceptable, but rather how much 'statistical' error there is in parameter estimates which reflect subjective opinion as much as they reflect data. Unsurprisingly, the error analysis shows the reliability of the calculated FS to be sensitive to two main parameters, ϕ_3 and K_0 . Uncertainty in the other parameters contribute negligible error.

Quality Control and Quality Assurance

The main purpose of field control testing during embankment construction is to ensure that the work performed complies with design requirements. Field control testing usually consists of density tests involving the determination of water content, dry density and classification of the compacted material. On the basis of these test results, the work performed within an area of concern

is either accepted or rejected. This section summarizes the detailed analysis of various aspects of the field density tests performed during construction of zone I of the Carters Dam.

Corps Specifications

Department of the Army Engineer Manual 1110-2-1911, "Construction Control for Earth and Rock Fill Dams," provides guidelines for field control testing of impervious materials. This manual suggests that compaction control tests be performed more frequently at the beginning of construction and reduced once the contractor and inspection personnel are familiar with material behavior and acceptable compaction procedures. It is further suggested that routine control tests be performed for every 1000 to 3000 cubic yards of compacted material.

Contract documents for construction of the Carters Dam specified that placement water content of zone I material be not more than 2% above or less than 2% below optimum moisture as determined by the contracting officer. No specifications were given for minimum acceptable dry density of the compacted material. The various types of equipment used for compaction and required number of passes were determined on the basis of test fill studies performed prior to construction. Nonconforming water content materials were corrected at the contractors expense. Nonconforming densities at the specified compactive effort were corrected at the Corps' expense.

Approximately 1.8M cubic yards of highly weathered and disintegrated rock was placed during construction of the main embankment core. The major portion of this impervious material was taken from borrow area number 3 (Figure 12). The resident Engineer Office inspection personnel performed 1244 sand cone

density tests on zone I material. Figure 42a and 42b show the location of most of these density tests. Figures 43, and 44 show the number of density tests per volume of compacted material and the date of each test, respectively. We could not find an explanation for the increased rate of testing after elevation 950'.

Statistical Description of Compaction Data

Histogram and probability plots of percent compaction and deviation from optimum moisture content ("±-percent water content") data of the accepted density tests are shown in Figures 45 and 46. The percent compaction data resemble a normal distribution while ±-percent water content data are somewhat truncated at +2%. These data were also plotted in Figure 47 as percent compaction vs. ±-percent water content. Approximately 6% of the accepted tests have a percent compaction less than 95% and 8% were either greater than or less than 2% optimum. About 10% of the density test data plot above the 100% saturation line as shown in Figure 48, which is consistent with the findings of other research (John Schmertmann, personal communication).

Figure 49 shows a rejection chart of the density tests (i.e., rejections recorded as a function of cumulative tests). The rejection rate (cumulative number of rejections/cumulative tests) during the early stages of construction was 20 to 30%, and slowly decreased to about 1% toward the end of construction. The data exhibit typical characteristics of learning curve effect, where control over a process improves with time. This is also shown in differences between the histogram of percent compaction of those test performed in power house excavation material and those performed in borrow area 3 material (Figure 50 and 51). The power house excavation material was placed during the early

stages of construction whereas borrow area 3 material was not used until one year after the start of construction. Although both data sets have the same mean (98.7%), the borrow source 3 material is more normally distributed and has significantly lower standard deviation (2.6 vs. 3.4). The decreasing trend in standard deviation of the percent compaction data and hence increase in construction control, is also shown in Figure 52, where standard deviation of \pm -percent water content (moving average cell = 20 tests) is plotted versus cumulative test number.

For the purpose of investigating whether significant trends exist in the compaction data, several moving average plots were constructed. The moving average \pm -percent water content data is shown in Figure 53. This plot seems to indicate that the data exhibit a trend which divides the plot into three zones. Histograms and probability plots of the three zones were constructed and are shown in Figures 54, 55, and 56 for the first, second, and third zones, respectively. These plots indicate that the data of zones one and three are normally distributed whereas the data of zone two appear to be truncated, somewhat resembling a uniform distribution.

Further analysis of the \pm -percent water content data displayed other interesting characteristics. Figure 57 shows the number of accepted density tests with \pm -percent water content greater than 2% (i.e., not within specification) versus total number of tests. During the first two years of construction of the main embankment the number of accepted density tests with water contents not within contract specifications was constant at a rate of 11%. However, between September 1967 and January 1969 only two such tests were recorded. After this the rate again reverted to 11%.

Most--but not all--of the these unacceptable water content measurements were coupled with high percent compaction values and were accepted. However, the change in data behavior during the interval September 1967 to January 1969 is note worthy.

The final stage in the compaction data analysis program was to investigate if density test results exhibit significant correlation structure. Vertical autocovariance among percent compaction values for those tests taken at the same stratum was computed. Although this is not a completely acceptable method of calculating the vertical autocovariance it was intended only for preliminary investigation purposes. The results for this analysis are shown in Figure 58, which indicates no significant vertical autocovariance in the data. After these computations were made the horizontal autocovariance among the density tests performed between elevations 980 and 999 was calculated and is plotted in Figure 59. Once again no significant autocovariance is shown in the data. However, when the autocovariance among density tests is calculated using 'test number' rather than spatial separation to represent lag distance (i.e, the lag between tests #1 and #2 would be 1) the data do display significant autocovariance (Figure 60). The bottom portion of this figure is a blow-up of the first 100 lag distances of the top graph. The implication of the clear autocovariance in the sequence data appears to be that the distance (or soil volume) within which significant correlation exists is smaller than the typical spatial separation of density tests and is perhaps a non-euclidian function of location within the compacted core. That is, the correlation follows the placement sequence (presuming this to be roughly congruent with the testing sequence) more than it does rectangular coordinates in the core geometry.

A variety of quality control and quality assurance techniques were applied

to the compaction control data from Zone 1 to assess whether such sampling technology, (a) would have streamlined the construction control procedures, and (b) would have led to timely identification of quality control situations such as the peculiarities in the compaction data indicated in Fig. 57. These applications are described in the report, "Statistical quality control for engineered embankments," (Contract Report GL-87-). The conclusion of these test applications was that statistical quality control and quality assurance techniques for construction inspection in dam projects appears to offer significant advantages to USAE projects, and these methods should be further investigated. For example, the peculiarities of Fig. 57 were immediately apparent when quality control data were plotted using the common technique of a cumulative-sums (QSUM) chart (Fig. 61).

Zonation of Core by Grain Size Distribution

In addition to compaction control data, grain size distributions were analyzed for as-placed material in zone 1. The locations of these grain size samples are shown in Figure 62. These data were used to evaluate piping potential between zone 1 and upstream zone 2 materials using now-developing statistical procedures. Such new procedures are only experimental, but the analysis of spatial pattern required as input to those methods in itself has led to interesting results.

Figure 62 is the result of a statistical clustering analysis of D₈₅ data from the grain size distribution tests. In this analysis, three principal clusters of similar grain size characteristics were isolated. These are numbered 1 to 3 in the figure, and located where the test specimens were sampled. The results show the existence of a subzone of relatively finer

material near the crest of the core, and pockets of relatively coarser materials near each of the abutments. A similar analysis of zone 2 upstream of the core is now in progress to assess the overlap of fine and coarse subzones in the two adjacent materials.

Continuing Directions

Results from the Carters Project application indicate that much can be learned about the progress of construction and its control from an analysis of compaction data or other quality control information. While statistical quality control and quality assurance techniques were only a small part of the present project, the potential usefulness of such techniques make it appear that further developmental work would be fruitful.

A convenient and powerful way to study these QC data is by use of various graphs and charts (e.g., successive histograms, probability plots, moving average trends, and so on). These graphs and charts afford a more comprehensive and clearer picture of the construction progress than tabulated numbers alone do. Another method of data analysis and presentation which may assist supervisory personnel in assessing construction process is statistical quality control charts. These have been used in manufacturing for many years. The development of simple control charts for percent compaction, water content and other significant performance data will enable inspection personnel to identify trends in the construction process, assist in trouble shooting when compaction control problems arise, and better carry out Quality Assurance responsibilities of the Contractor QC program.

The use of cheaper and faster, but noisier, measurement tools for compaction control needs to be investigated. For example, the use of nuclear

gages rather than sand cone tests to determine in situ density would provide more data at the same cost and would allow more efficient statistical planning. Whether more numerous but lower quality measurements provide better overall control remains an issue to be considered.

Other issues which deserve attention are the comparative efficiency and accuracy of random vs. nonrandom sampling plans, whether acceptance of compaction work should be based on 100% compliance with specifications or on a specified fraction of tests passing a higher performance specification, and whether the sampling of compaction should be based on attributes or variables.

Criteria for Comparisons

Three criteria were identified against which to assess the benefits of statistical methods for geotechnical engineering aspects of new dam projects, as compared with present procedures. These were, productivity, quality control, and technologic capability.

Productivity as used here refers to the efficiency with which man-power is used in engineering and related activities of dam projects. Productivity can have to do with the number of data that can be analyzed with a given number of man-months effort, or with the commitment of engineering time necessary to accomplish certain design activities, or with the number of field inspections necessary to control fill placement, or so on.

Quality control refers to the maintenance of reliability in engineering analysis, construction operations, or other activities related to geotechnical engineering aspects of a project. Reliability in this sense means confidence in the repeatability of results, whether those results be calculations,

in-place properties of engineering fills, or other products of engineering and construction.

Technologic capability means what is technologically possible for geotechnical engineers to achieve on a dam project. Technologic capability can refer to the ability to model physical phenomena, the ability to collect or analyze data of certain types, the ability to control construction operations, and so on.

Four other criteria were considered in evaluating the usefulness of individual statistical methodologies:

1. Is the methodology tested?
2. Is it understandable?
3. Does it provide useful results?
4. Does it promote computerization?

Consideration of productivity benefits of new technology for dam engineering or construction requires identification of where engineering effort is spent on projects, and the questions for which finding answers occupies a great deal of time and effort. On the Carters Project, and perhaps on most projects, finding answers to the question which arguably had the most influence on design and cost did not occupy the bulk of engineering time.

Major Trends in Engineering and Construction

Two major trends in engineering and construction exert influence on any decision the USAE might make regarding statistical methodology. The first is rapid computerization. The second is increased regulatory oversight. Neither of these appear strongly in the retrospective analysis of the Carters Project,

but each changes the environment within which future USAE projects will be undertaken. Each increases the usefulness and applicability of statistical methods for geotechnical engineering aspects of dam projects.

Computerization

No one doubts the profound impact of technologic advances in computerization on civil engineering. Engineering and construction organizations such as the USAE cannot avoid integrating computer capabilities into their operations because productivity benefits will require it. Indeed, the USAE has already embarked on a major program of computer capability development for geotechnical engineering. The more obvious changes this computerization will bring to geotechnical engineering are,

- (a) Larger data sets.
- (b) More extensive modeling and analysis.
- (c) Integration of engineering and construction data bases.
- (d) Real-time feedback of facility performance to engineering analysis.

Each of these changes requires data to be manipulated and managed via computer programs. Such programs will not have the capability to intuitively describe, summarize, and draw inferences from data, but must rely on formal algorithmic procedures. Statistics is the branch of mathematics from which these procedures come.

By today's standards, many statistical procedures are cumbersome and difficult to implement. A user must understand details of mathematical procedures which, while important to calculations, are unimportant to understanding the meaning of results or to intelligently use result.

Regulatory Oversight

The second major trend is increased regulatory involvement in civil engineering projects. This increase reflects among other things, (a) increased concern for public safety aspects of civil projects, especially large projects such as dams, and (b) change in the mix of projects large engineering organizations face, especially the increase of hazardous waste facilities and decaying infrastructure. In contrast to the informal, intuitive approach to geotechnical engineering design which has characterized traditional practice, regulatory authorities demand demonstratable assurance on (a) sources, reliability, and analysis of data; (b) defensible connections between facility performance predictions and supporting data; and (c) quality control in engineering and construction.

In each of these three areas, present practice does not provide a viable foundation for providing assurance to regulatory authorities. Regulatory activities require explicitness and tracability. One way to satisfy this need, perhaps the only practical way, is with statistical procedures.

Geotechnical engineers--often with good reason--find the regulatory approach to project design misdirected. Most geotechnical engineering requires substantial judgement, and design evaluations which can only be made while a project is in construction. This requirement has led to professional procedures epitomized by the observational method so widely practiced on dam projects. Nonetheless, geotechnical engineering practice must coexist with regulatory requirements, and to this end, statistical methodology is a helpful tool.

Assessment of Present Statistical Methodology for Geotechnical Engineering Aspects of New Dam Projects

This section summarizes assessments made about the present state of statistical methods for geotechnical aspects of new dam projects, based on the applications of those methods to the Carters Project data. The question of concern in the Carters Project applications has been to what extent statistical methods would have changed--presumably improved--activities related to the design and construction of that specific project. Geotechnical engineering aspects of future dam projects will differ in many ways from those of the Carters Project. These new conditions clearly cannot be ignored in decisions about new technology. Thus, the assessment of statistical methodology in the present section attempts to combine experiences gained through the case study with forecasts about the future of geotechnical engineering.

The assessment of statistical techniques that were applied to the Carters Project data is summarized in Tables 5, 6, and 7. In these tables the techniques are judged on two dimensions: their apparent usefulness or benefit, and the maturity of their technological development. Tables 5 and 6 show a somewhat detailed breakdown of specific methods considered during the study. Table 8 attempts to group the statistical methods into broader classes. Techniques which appear to the top and right of Table 8 are those methods which appear to offer benefit and which are already developed to the point of practical usefulness. These are the methods that are more fully discussed in accompanying guides. Those methods which appear to the bottom and left of Table 7 are those whose benefits may be limited, and which require further developmental effort before they will be useful in practical applications.

The rankings shown on Tables 5, 6, and 7 are subjective. They represent best estimates based on the experience of applying the methods to the Carters Project data base. These rankings would probably change somewhat had a project other than Carters been used as the case study, and they would probably change somewhat if other people were making the judgements.

The conclusion from this ranking is that at least three categories of statistical methods appear to offer benefit to geotechnical engineering aspects of new dam projects, and are also sufficiently developed that they might be introduced to practice. These are, (1) routine statistical analysis of geotechnical test data, (2) error analysis of geotechnical calculations, and (3) statistical quality control and quality assurance for construction inspection of engineered fills.

Table 3: SPT Summary Statistical Data for Reregulation Dam Foundation

CALCULATION SHEET

PROBLEM: Rereg blow count profile
 DATE: 6/02/84

CALCULATED BY: C.E.
 CHECKED BY: G.B.

DESIGN PROFILE: Reregulation Dam

(1) DATA SCATTER: SPT

Station	4+00--13+00	17+00--24+50	25+00--32+00
mean (bpf)	4.8	6.9	8.9
standard deviation	2.9	2.8	4.4
coefficient of variation	0.60	0.41	0.49
Measurement Noise (From Figures 19, 20)	--	--	--
Spatial Variability $\sqrt{V[x]} = \sqrt{(V[z]-V[e])}$	2.9	2.8	4.4

(2) SYSTEMATIC ERROR

Station	4+00--13+00	17+00--24+50	25+00--32+00
number measurements* per depth interval	14	11	20
Statistical Error $\sqrt{V[m_x]} = \sqrt{V[z]/n}$	0.78	0.84	0.98
Model Bias	n/a	n/a	n/a
Total Systematic Error	0.78	0.84	0.98

* (varies with depth, numbers are representative)

(3) DESIGN PROFILE

(Shown as Figures 28a,b, and c)

Table 4

LABORATORY TEST DATA TALLY

Laboratory Tests	MAIN DAM							REREGULATION DAM				
	Tests Fills						Zone 1 undistd. after plcent.	Fndn. Soils from SS borings	Test Fills		Borrow Areas	
	#1	#2	#3	#4	#4	#5			A*	B*	C*	D*
gradation att. limits specific gravity	2	2	2	8	14	4	54	51	9	4	6	1
standard compaction	-	-	-	-	2	-	54	-	-	4	3	1
permeability	-	-	-	-	1	-	41	9	-	-	-	-
consolida- tion	-	-	-	2	2	-	54	29	3	42	3	2
strength tests												
"Q"(UU)	1	1	1	2	5	-	54	24	3	4	12	4
"R"(CU)	1	1	1	2	5	8	54	24	3	4	12	4
"S"(DS)	-	-	-	2	4	4	54	21	3	4	12	4

*Test Fill "A": Foundation materials from reregulation dam

Test Fill "B": Embankment materials from reregulation dam

Borrow Area "C": Channel borrow

Borrow Area "D": Borrow areas 1 and 2

Table 5. Reregulation Dam Stability Calculation

CALCULATION SHEET

PROBLEM: sudden drawdown Sta 6+50

CALCULATED BY: C.E.

DATE: _____

CHECKED BY: G.B.B.

(1) RAW DATA

- a) For cross section 6+50 sudden drawdown conditions.
b) R test data -- Borrow Areas 1 & 2 (impervious fill):

Remolded sample	c(TSF)	ϕ	$\tan \phi$
-2% w _{opt} , 95% Y _{dmax}	0.58	16.5	0.296
w _{opt} , 95% Y _{dmax}	0.60	17.0	0.306
-2% w _{opt} , 100% Y _{dmax}	0.25	20.5	0.374
+4% w _{opt} , 100% Y _{dmax}	0.75	15.5	0.277

$$\begin{aligned} m_c &= 0.55 \\ s^2_c &= 0.044 \end{aligned}$$

$$\begin{aligned} m_{\tan \phi} &= 0.313 \\ s^2_{\tan \phi} &= 0.002 \\ C[c, \tan \phi] &= 0.009 \end{aligned}$$

- c) Constants:

Length, L = 105.7 feet
sum tangential forces, $\Sigma F_t = 108.9$

(2) CALCULATE EXPECTED VALUE OF FS (Figure 37)

$$m_{FS} = \frac{L m_c + N_1 m_{\tan \phi 1} + N_2 m_{\tan \phi 2}}{m[\Sigma F_t]} = 2.07$$

$$\begin{aligned} s^2_{FS} &= \left(\frac{L}{\Sigma F_t} \right)^2 s^2_c + \left(\frac{N_1}{\Sigma F_t} \right)^2 s^2_{\tan \phi} + \left(\frac{L}{\Sigma F_t} \right) \left(\frac{N_1}{\Sigma F_t} \right) C[c, \tan \phi] \\ &= (0.105)^2 \end{aligned}$$

(3) RELIABILITY INDEX

$$\beta = \frac{m_{FS} - 1.0}{s_{FS}} = \frac{2.07 - 1.0}{0.105} = 10$$

$$\beta = \frac{m_{FS} - 1.5}{s_{FS}} = \frac{2.07 - 1.5}{0.105} = 6$$

Table 6. Main Embankment Stability Calculation

CALCULATION SHEET

PROBLEM: wedge stability
main embankment

CALCULATED BY: A.L.
 CHECKED BY: G.B.B

Parameters:Equation:

	Best Estimate	Variance
WNB1	3,817,000 #	1,457,000 # ²
WNB2	20,274,000 #	10,276,000 # ²
tan ϕ_2	tan 30°	4.1E-4
tan ϕ_3	tan 35°	7.6E-5
h	447.3 ft	---
h _w	404 ft	---
γ	71 pcf	0.05 pdf ²
γ_w	62.4 pcf	---
K _O	0.65	1.4-3

$$FS = \frac{WNB1 \tan \phi_1 + WNB2 \tan \phi_2}{(0.5)h^2 K_O \gamma + (0.5)h_w^2 \gamma_w}$$

Best Estimate: $m_{FS} = \frac{3817k \tan 30^\circ + 20274k \tan 35^\circ}{(0.5)(71)(447.3)^2(0.65) + (0.5)(62.4)(404)^2} = 1.66$

Uncertainty:

	Derivative ² *	Variance	Variance Contribution	%
<u>Numerator:</u>				
WNB1	0.33	1,457,000	480	0.3
WNB2	0.23	10,276,000	2363	0.8
tan ϕ_2	3817k ²	4.1E-4	1107	0.6
tan ϕ_3	20274k ²	7.6E-5	168524	43.0
			$\Omega^2 = 6E-4$	44.7
<u>Denominator:</u>				
γ	4.2E3	0.05	21	0.3
K _O	5.0E7	1.4E-3	70600	55.0
			$\Omega^2 = 7.5E-4$	55.3

* square of derivative of FS with respect to each parameter.

$$\Omega_{FS}^2 = (0.04)^2 \quad 100.0\% \quad \text{=====}$$

Reliability Index (for FS=1.5 criterion, and FS=1.0 criterion):

$$\beta = \frac{m_{FS} - 1.5}{s_{FS}} = \frac{1.66 - 1.5}{1.66(0.04)} = 2.4 \quad \beta = \frac{m_{FS} - 1.0}{s_{FS}} = \frac{1.66 - 1.0}{1.66(0.04)} = 10$$

Table 7

Potential Applications of Statistical Methods to Site Characterization

	MATURITY			POTENTIAL BENEFIT		APPLICABILITY	
	I 5-10yr	II 2-5 yr	III now	Low	High	Special Case	Routine
<u>Analysis of Spatial Variability</u>							
Regression and trend analysis			•	•			•
Interpolation and contour mapping (including Kriging)			•	•			•
Discrete mapping and discriminant analysis			•	•			•
Statistical pattern recognition		•	•	•			•
Rock joint surveys			•		•		•
<u>Uncertainty Analysis</u>							
Second-moment analysis of soils data			•		•		•
Estimation of measurement noise		•	•	•			•
Statistical design profiles		•			•	•	
Identification of outliers				•			
<u>Hypothesis Testing</u>							
Homogeneity of data sets			•	•			•
<u>Optimal Allocations of Exploration Effort</u>							
Search and detection of anomalies			•		•		•
Optimizing boring network geometry		•			•	•	
Optimizing numbers of tests		•			•		•
Optimizing mix of test types		•			•		•
<u>Miscellaneous</u>							
Statistical data base management		•	•	•			•
Correlation analysis of engineering properties				•	•		•

Table 8

Potential Applications of Statistical Methods to Engineering Analysis and Performance Monitoring

	MATURITY			POTENTIAL BENEFIT		APPLICABILITY	
	I 5-10yr	II 2-5 yr	III now	Low	High	Special Case	Routine
<u>Error Analysis of Engineering Predictions</u>							
Soil slope analysis			•	•	•	•	•
Rock slope analysis			•	•	•	•	•
Deformation modeling of embankments and foundations	•	•		•	•	•	•
Seepage and groundwater flow analysis		•					
<u>Numerical Models</u>							
Stochastic finite element analysis		•		•		•	
<u>Advanced Reliability Analysis</u>							
Level II reliability		•		•	•	•	•
Systems (multimode) reliability	•						
<u>Engineering Risk Analysis</u>							
Risk assessment for individual failure modes			•		•		•
Systems risk assessment		•			•		•
Consequence analysis		•			•		•
Economic risk/benefit analysis		•			•		•
Multiatribute utility theory			•	•			
<u>Miscellaneous</u>							
Statistical observational methods		•			•		•
Statistical liquefaction analysis	•					•	

Table 9
Potential Applications of Statistical Methods to Quality Control and Quality Assurance
for Construction Inspection

	MATURITY			POTENTIAL BENEFIT		APPLICABILITY	
	I 5-10yr	II 2-5 yr	III now	Low	High	Special Case	Routine
Quality Control Chart Techniques							
Control charts for mean, range, etc.			•		•		•
Cumulative reject charts			•		•		•
Moving average and range charts			•		•		•
Cumulative sum charts			•		•		•
Frequency distribution techniques		•					
Acceptance Sampling							
Acceptance sampling for fraction defective			•		•		•
Acceptance sampling for mean			•		•		•
Optimizing numbers of QA tests		•			•		•
Optimizing mix of test QA tests		•			•		•
Sequential Sampling Techniques							
Two stage sampling plans		•			•		•
Sequential sampling plans		•			•		•

Table 10

Current State of Development and Potential Usefulness of
Various Statistical and Probabilistic Techniques for Geotechnical
Engineering Aspects of New Dam Projects.

P O T E N T I A L I M P A C T

	MINOR or SPECIALIZED	MODEST	WIDESPREAD and IMPORTANT
READY NOW	-Search theory -Soil slope reliability	-Stochastic mapping -Statistical joint surveys -Multiatribute utility -Rock slope reliability	-Statistical data analysis -Error analysis of <i>G_r</i> calculations -Statistical quality control -Acceptance sampling for QA
	-Level II Reliability -Stochastic finite element modeling	-Statistical observational methods and updating -Estimation of measurement noise -Identification of outlier data	-Statistical data base mgt. -Decision theory for explor- ation planning -Subjective probability quantification -Expert systems for data analysis -Economic risk/benefit analysis
LONG-TERM (5-10 years)		-Statistical filter criteria	-Comprehensive (systems) risk assessment -Stochastic piping models -Statistical pattern recog- nition for ADP of geo- logical data

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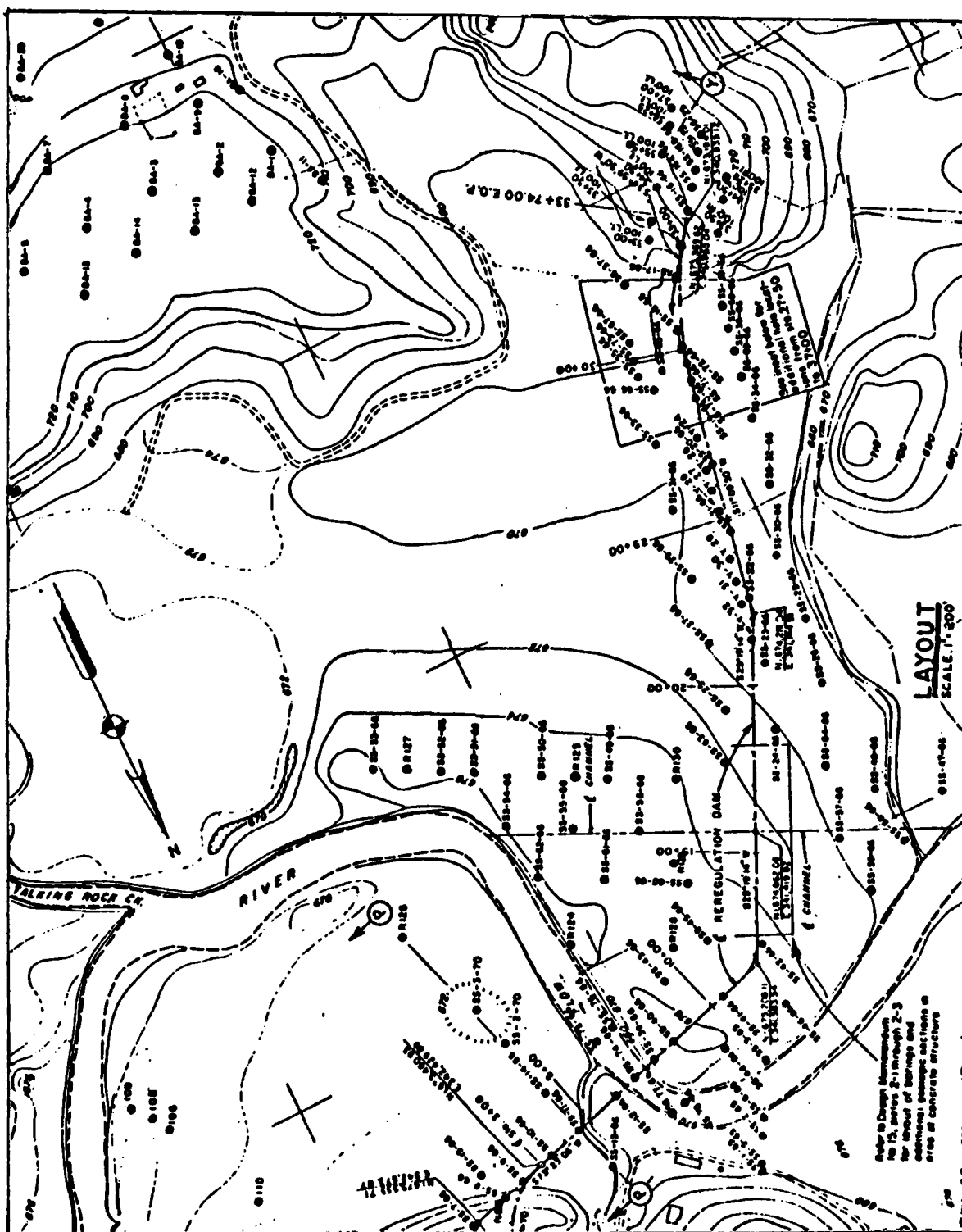
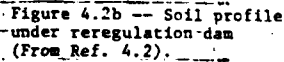


Figure 14. Boring Layout at Reregulation Dam.



120

TOTAL NUMBER OF DATA : 188
 MEAN : 4.8457 STD DEV : 2.8851
 NUMBER OF OUTLIERS BEYOND LOWER LIMIT OF PLOT : 0
 NUMBER OF OUTLIERS BEYOND UPPER LIMIT OF PLOT : 0

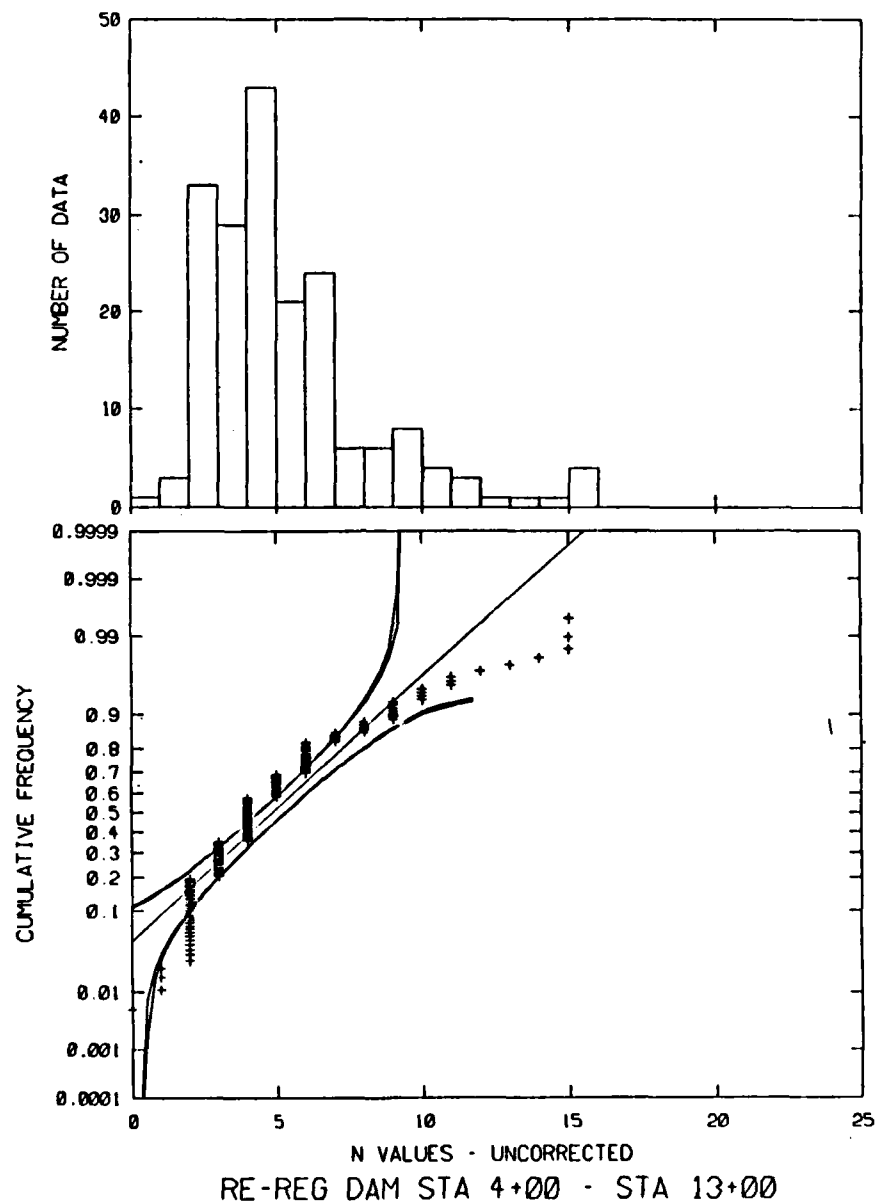


Figure 16. SPT Data At Reregulation Dam Site, Station 4+00 to 13+00.

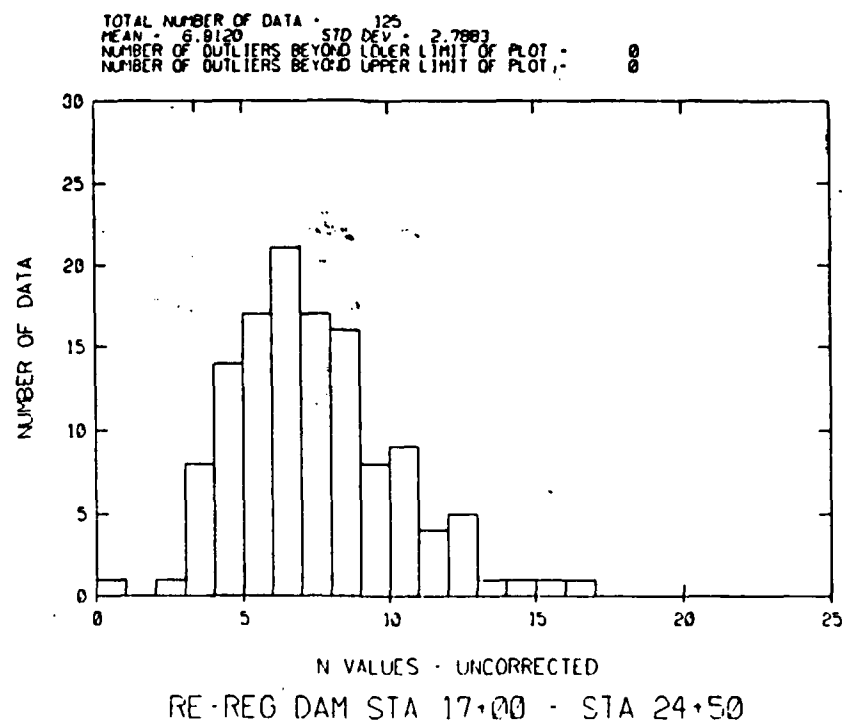
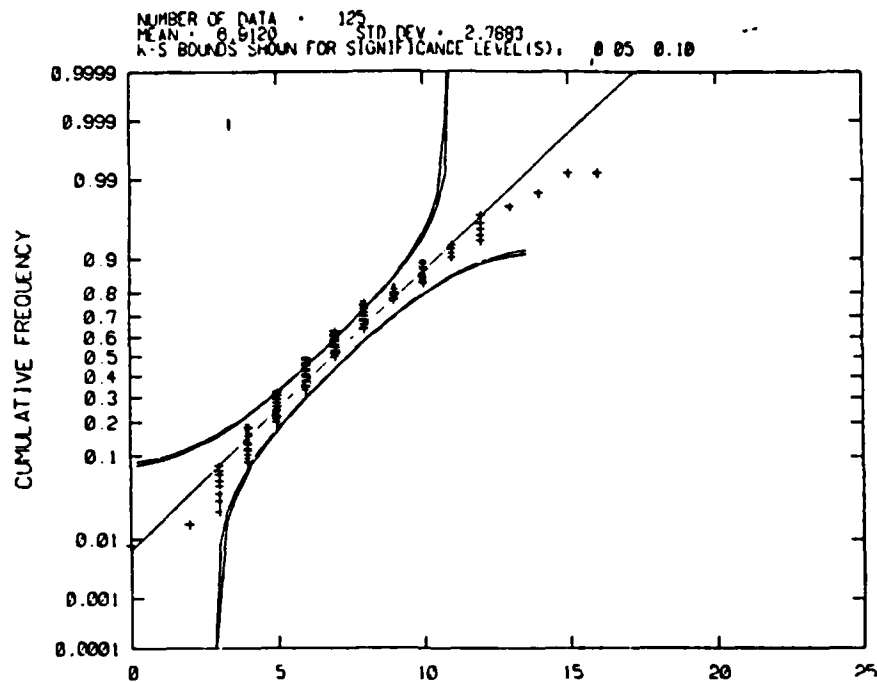


Figure 17. SPT Data At Reregulation Dam Site, Station 17+00 to 24+50.

TOTAL NUMBER OF DATA : 235
 MEAN : 9.5321 STD DEV : 4.4283
 NUMBER OF OUTLIERS BEYOND LOWER LIMIT OF PLOT : 0
 NUMBER OF OUTLIERS BEYOND UPPER LIMIT OF PLOT : 0

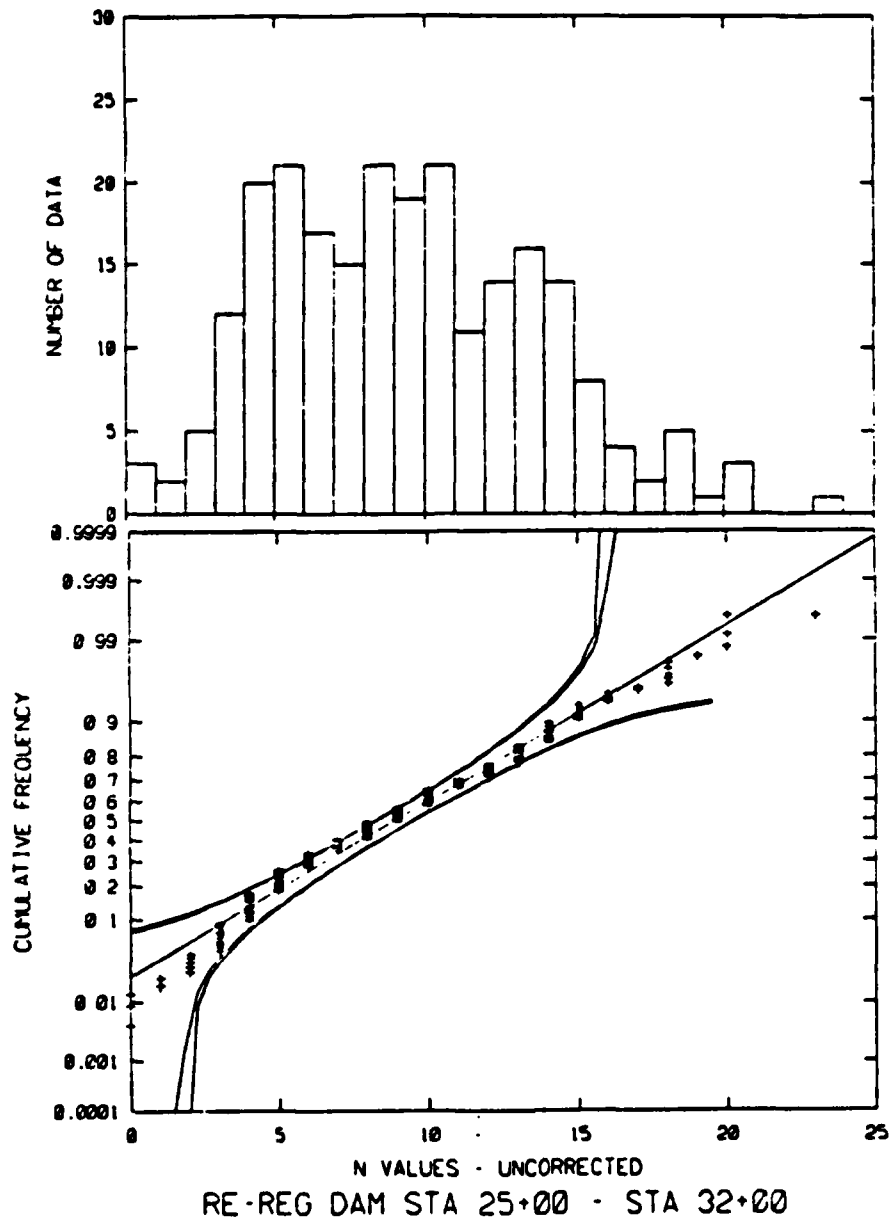


Figure 18. SPT Data At Reregulation Dam Site, Station 25+00 to 32+00.

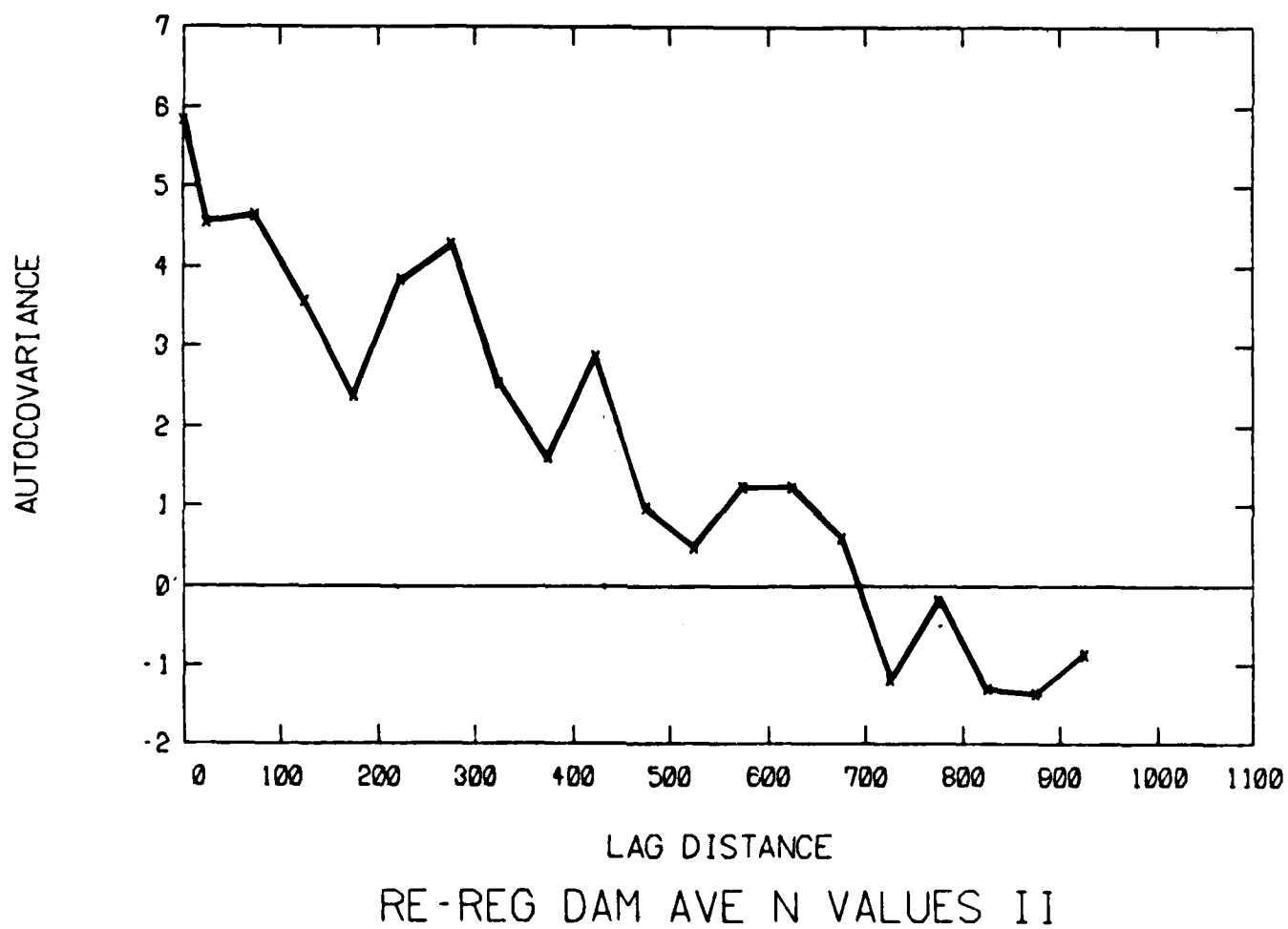
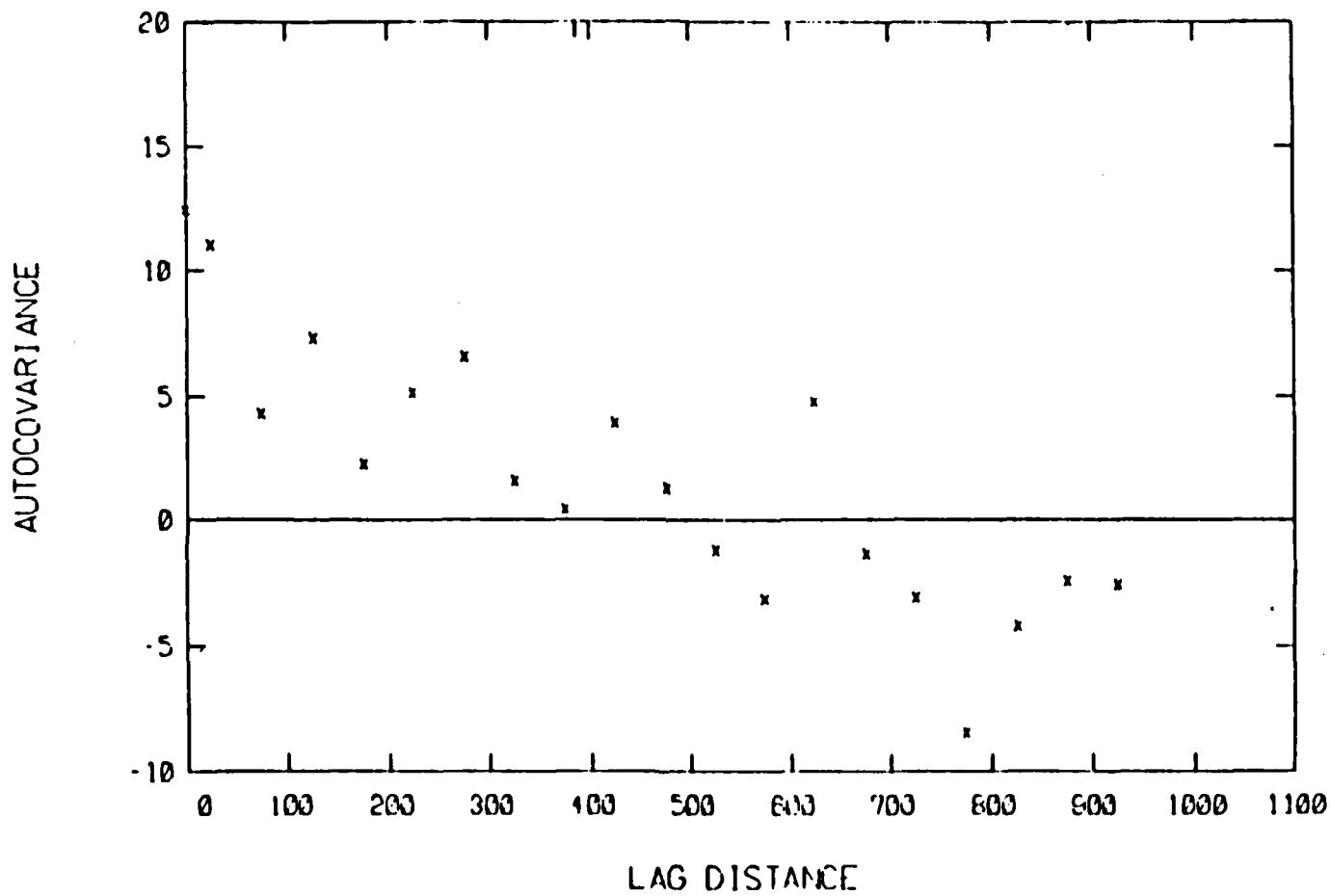


Figure 19. Autocovariance Function for SPT Data of Figures 16 to 19.



RE-REG DAM N VALUES EL 660.

Figure 20. Autocovariance Function for SPT Data at Elevation 660'.

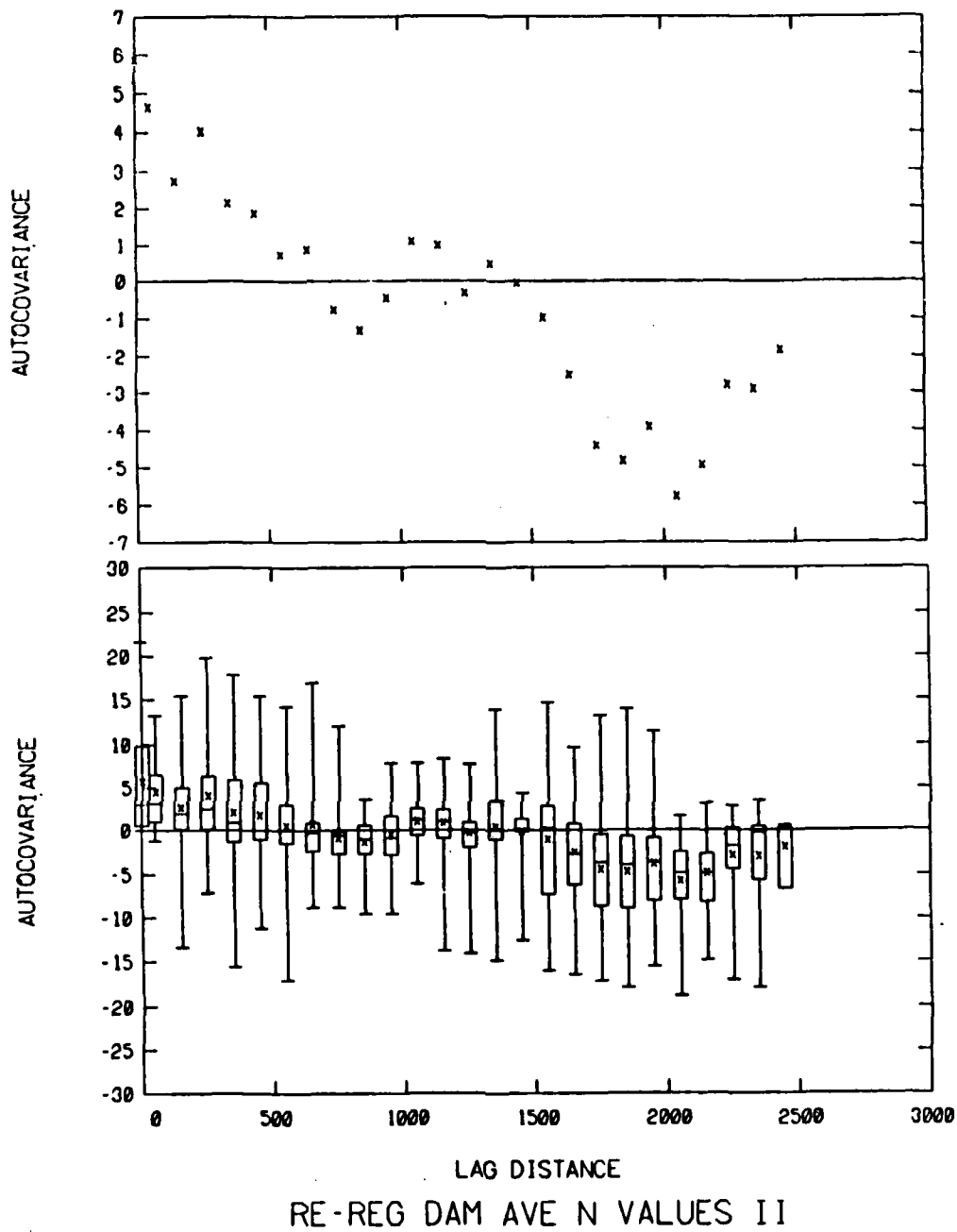


Figure 21. Autocovariance Function Estimated by Moment Estimator.

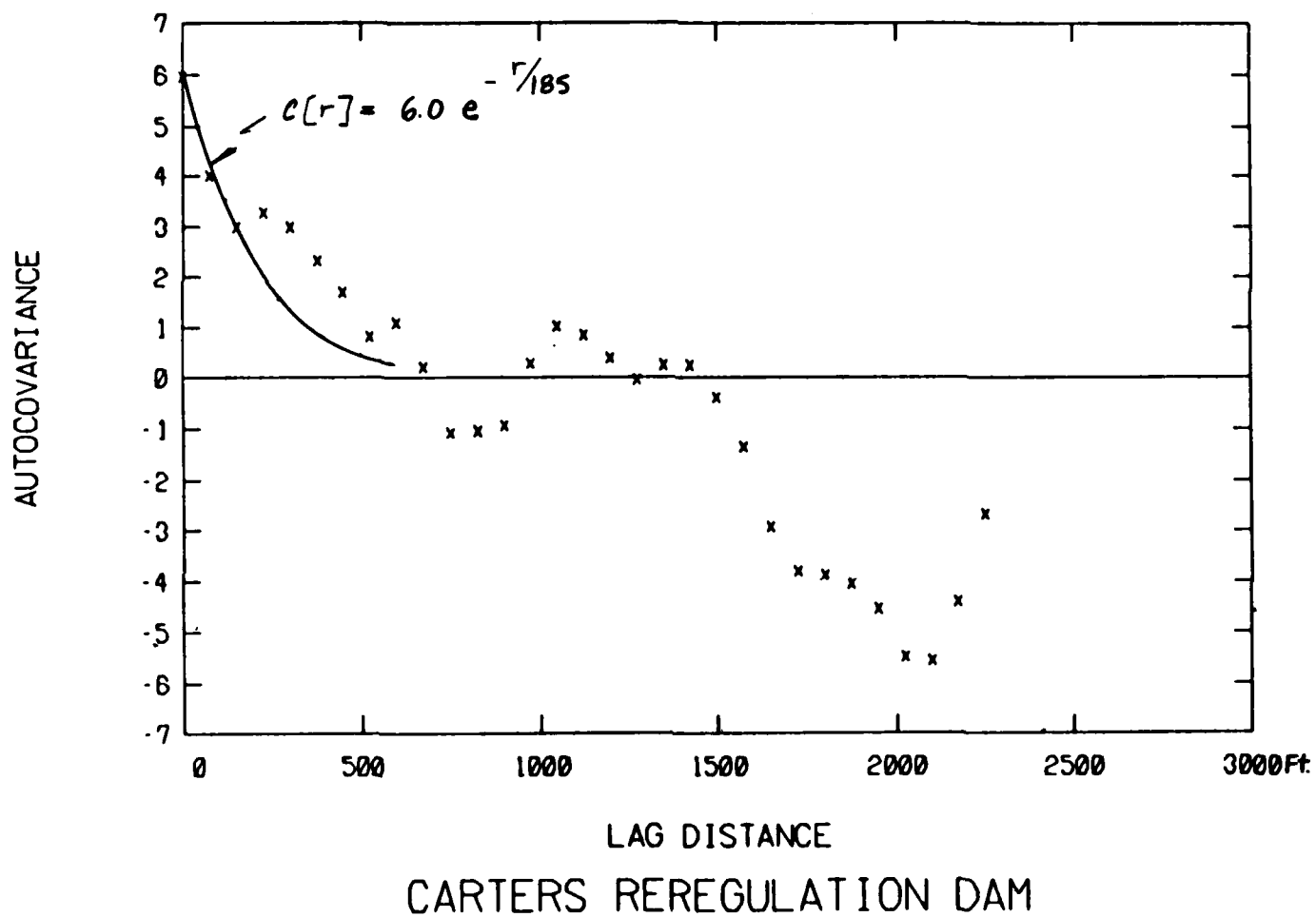


Figure 22. Autocovariance Function Estimated by BLUE Estimator.

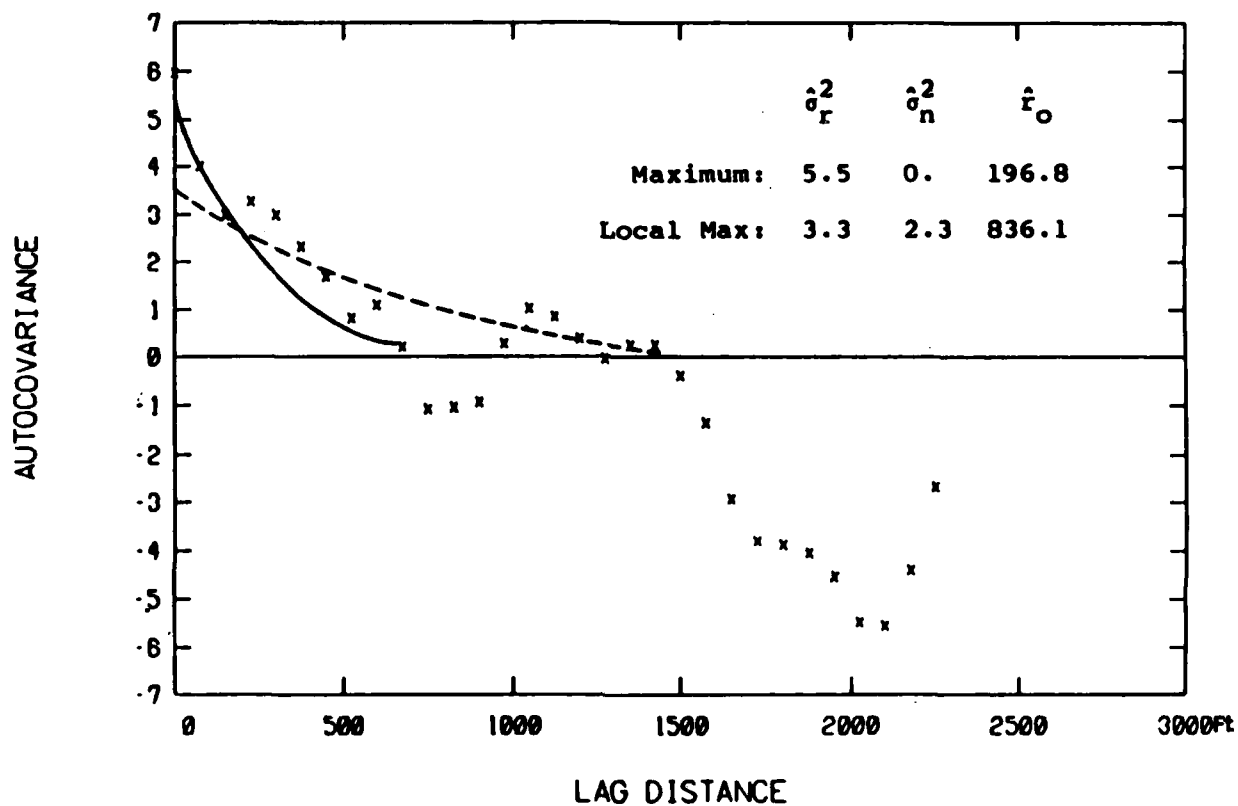


Figure 23. Autocovariance Function Estimated by Maximum Likelihood Estimator.

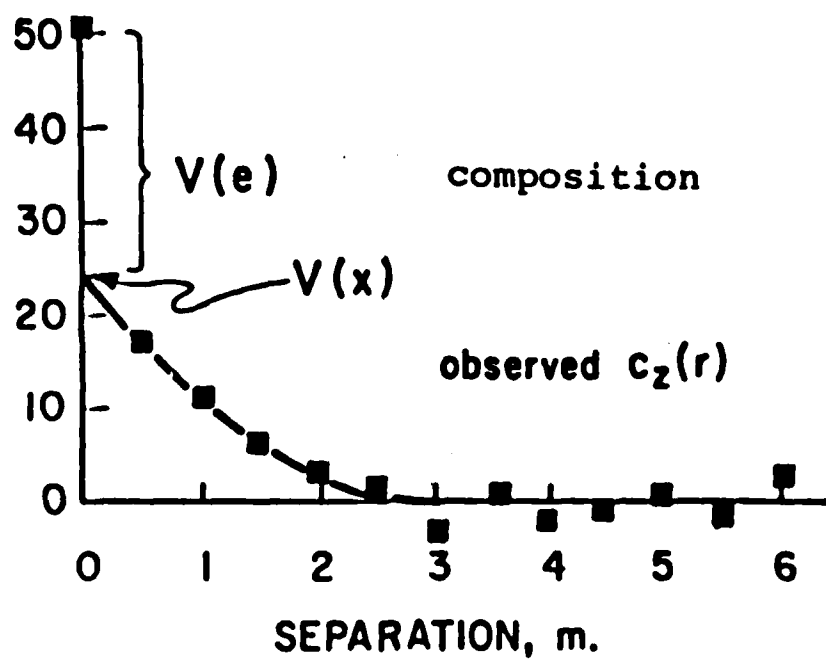


Figure 24. Composition of Autocovariance Function.

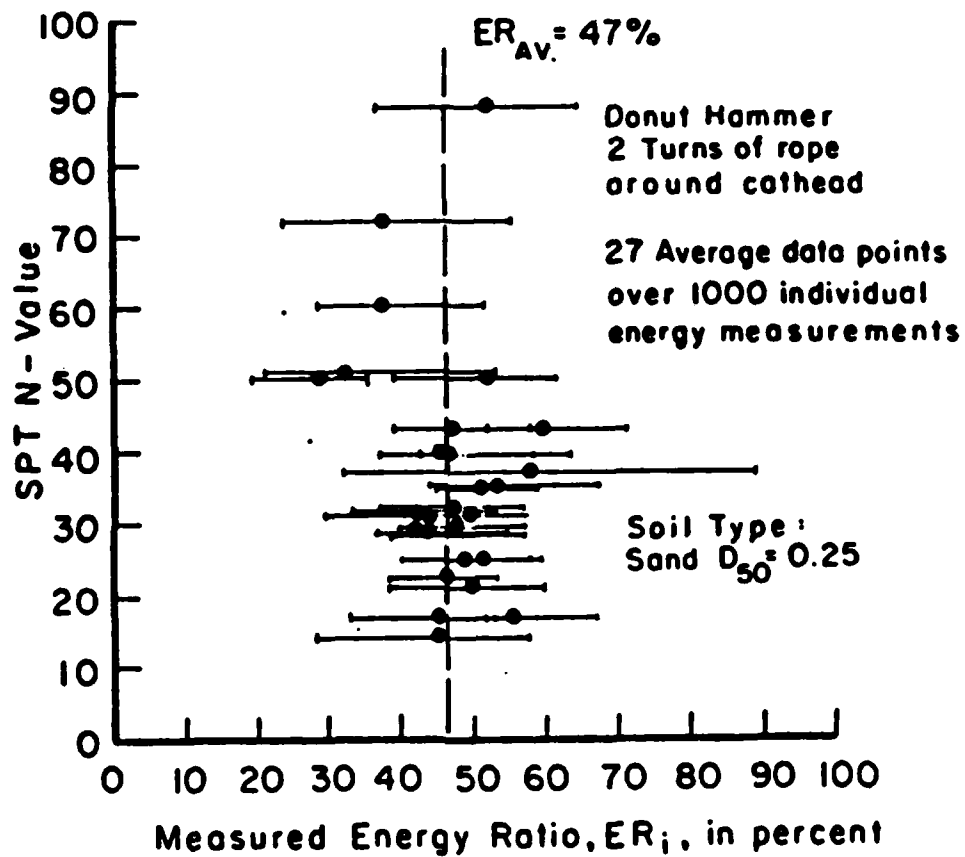


Figure 25a. Variability of Energy Ratio in SPT Testing (from Campanella, et al., 1984).

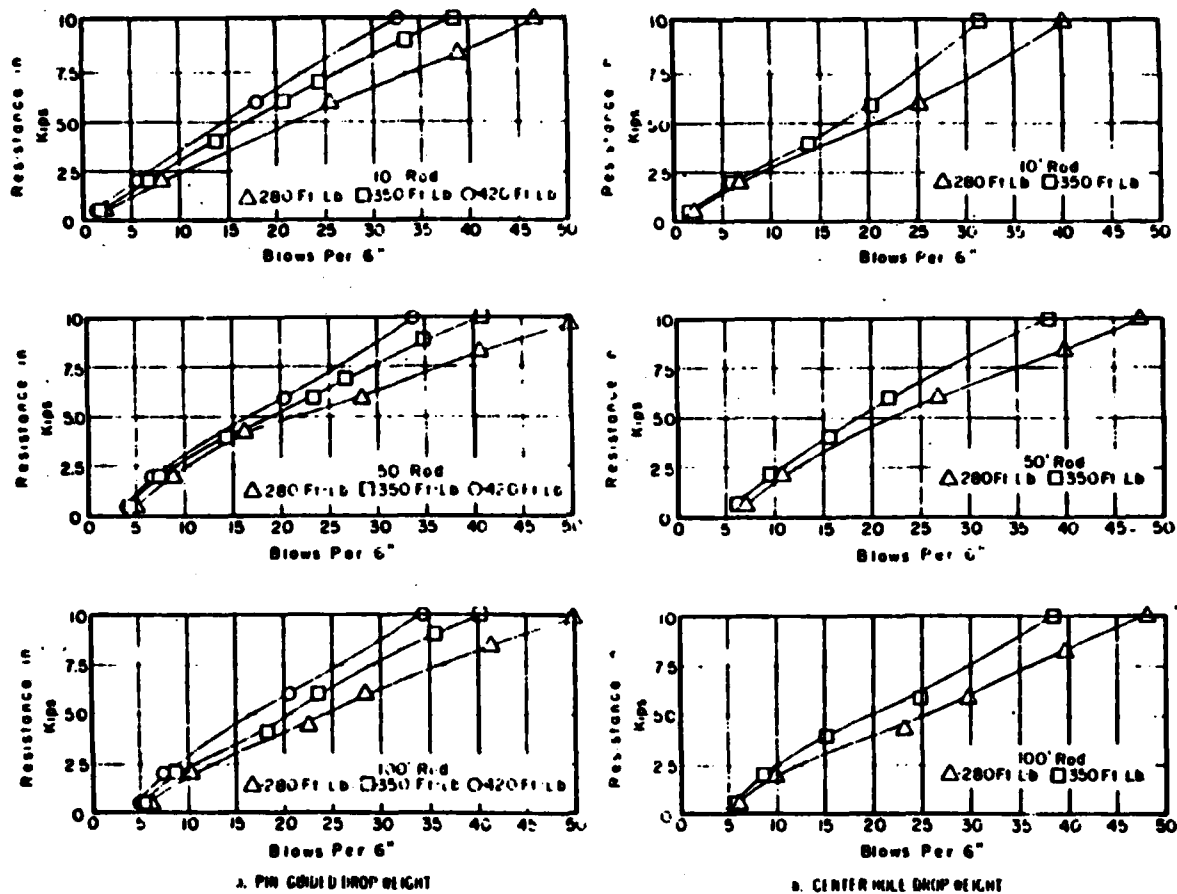


FIG. 4. INFLUENCE OF VARIATION OF HAMMER ENERGY ON THE SPT, IN TERMS OF BLOW COUNT VERSUS TOTAL RESISTANCE

Figure 25b. Sensitivity of SPT Blow Count to Energy Ratio (from McLean, et al., 1975).

TOTAL NUMBER OF DATA : 45
 MEAN : 0.1911E-01 STD DEV : 1.0051
 NUMBER OF OUTLIERS BEYOND LOWER LIMIT OF PLOT : 0
 NUMBER OF OUTLIERS BEYOND UPPER LIMIT OF PLOT : 0

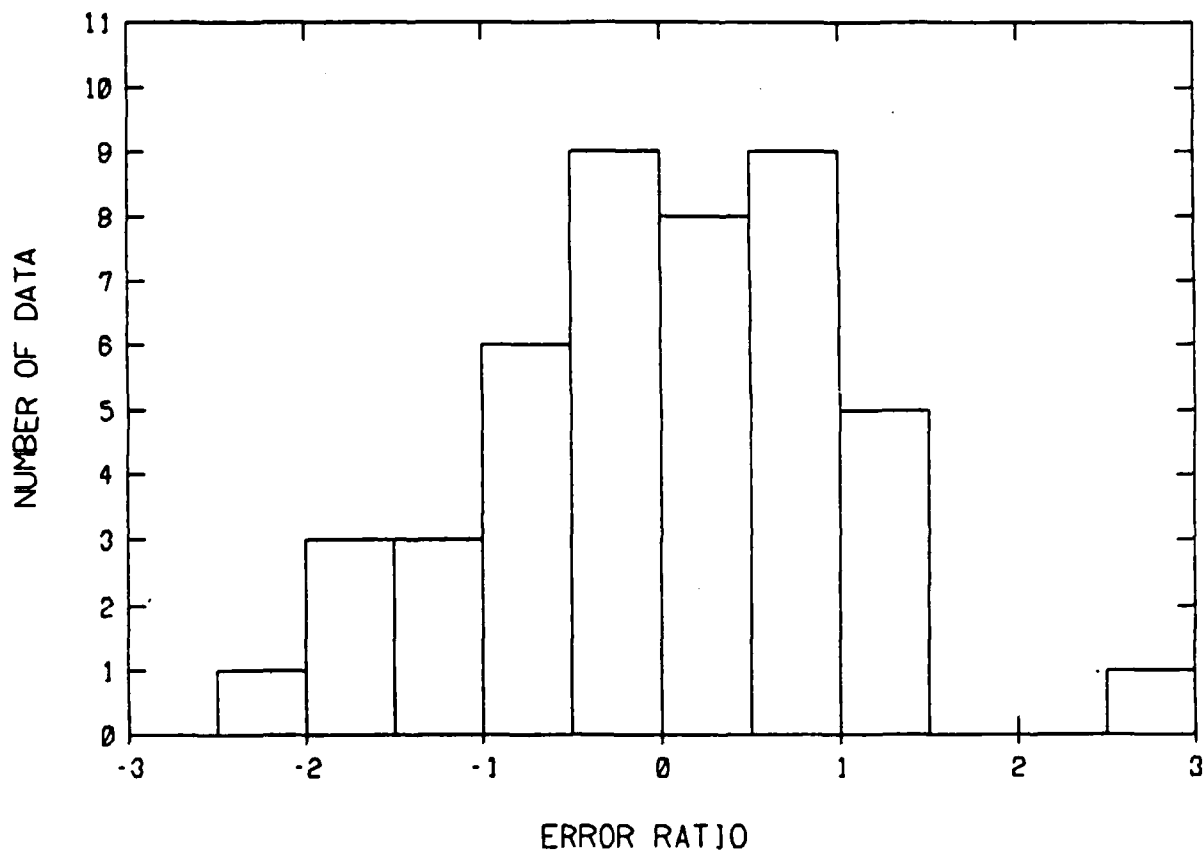


Figure 26. Statistically Estimated vs. Observed Data at Reregulation Dam, Showing Normalized Error Ratio.

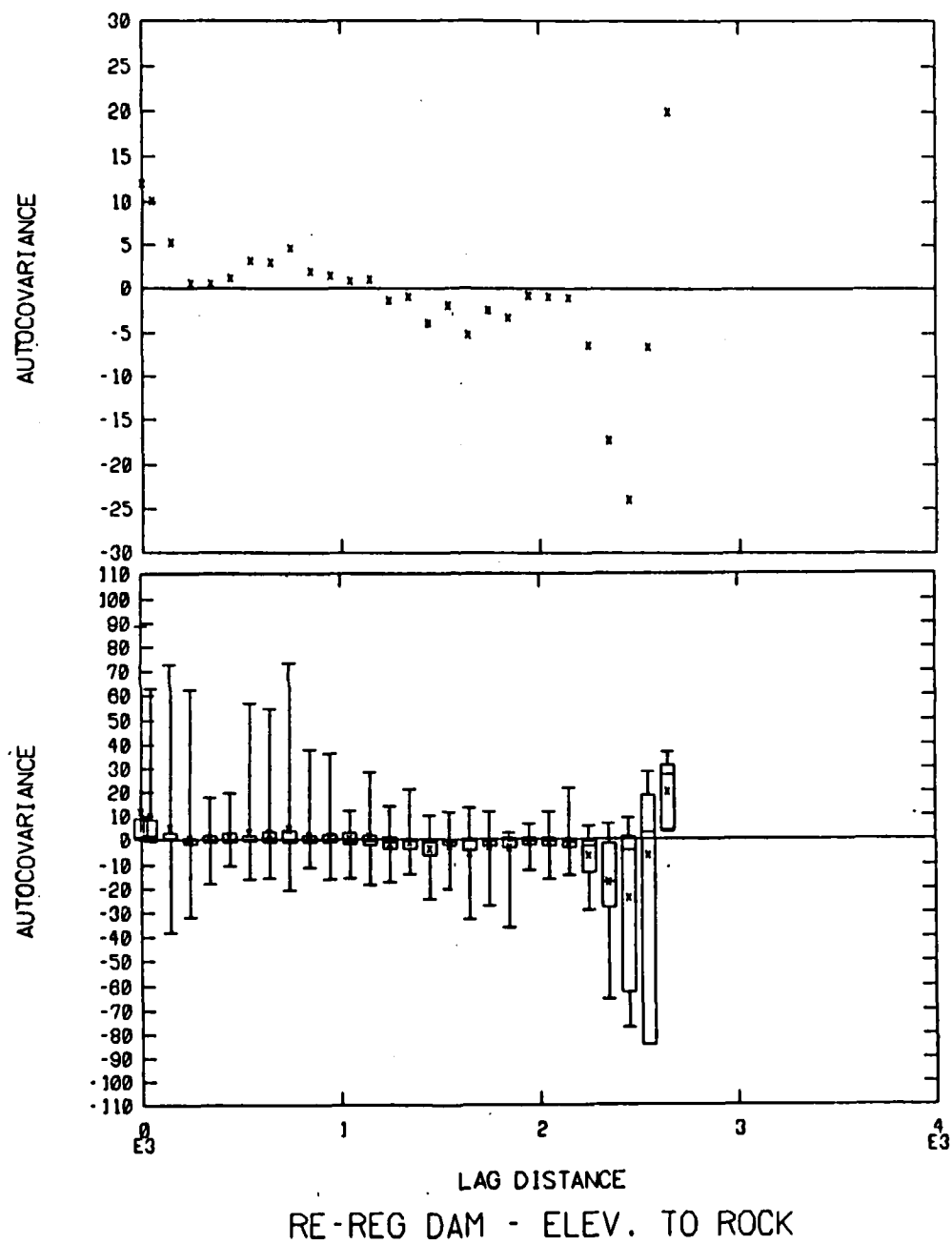


Figure 27. Autocovariance Function of Depth to Rock.

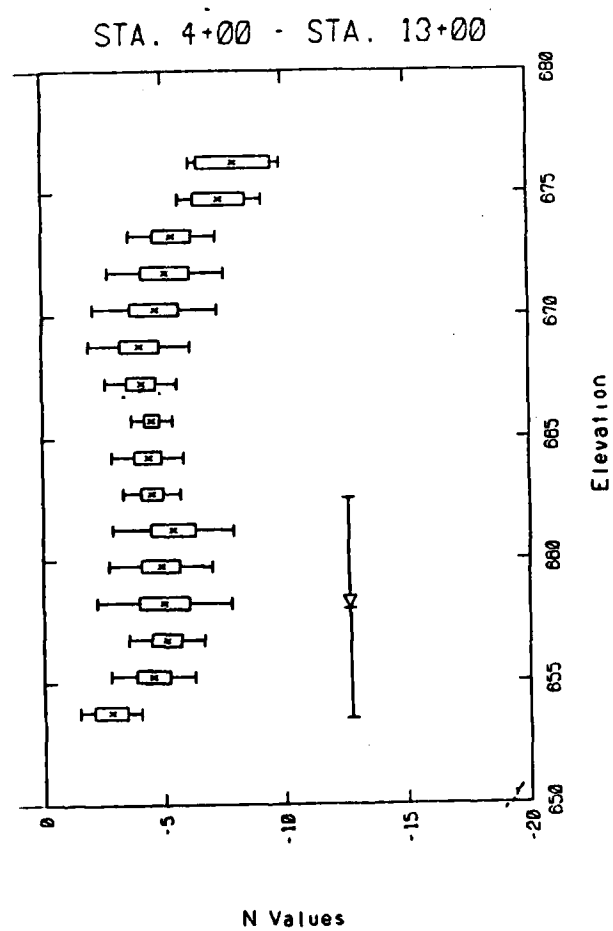


Figure 28a. Design Profile Station 4+00 to 13+00.

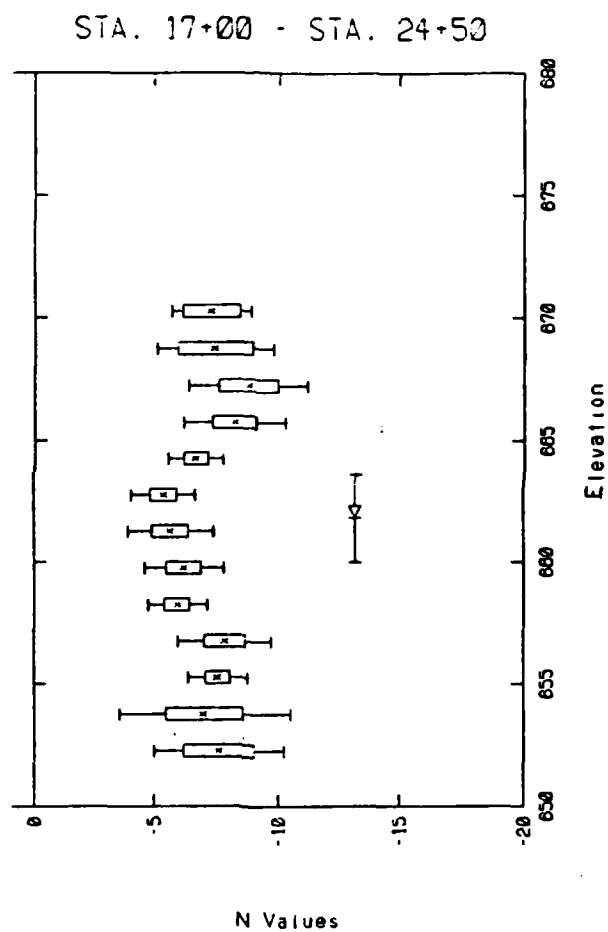


Figure 28b. Design Profile Station 17+00 to 24+50.

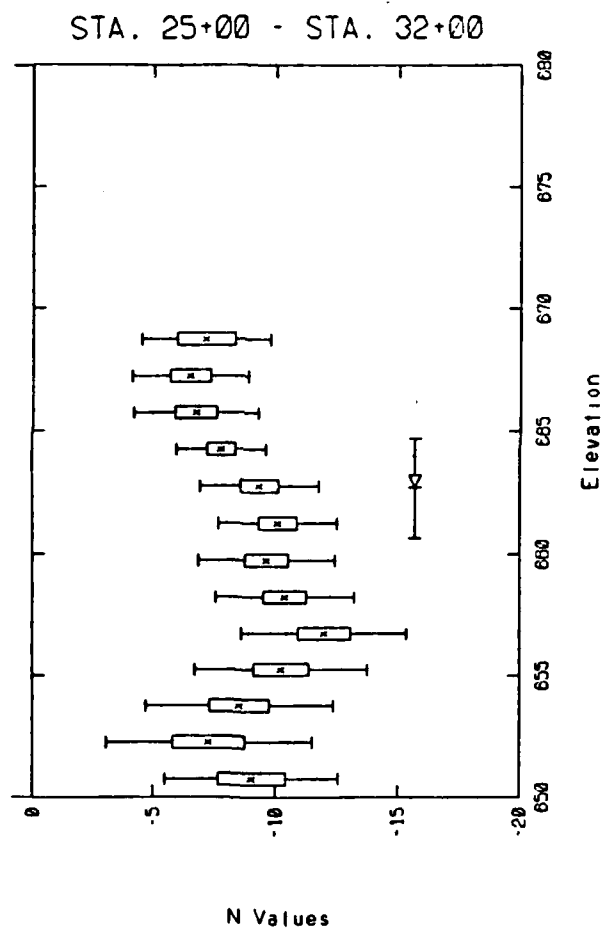


Figure 28c. Design Profile Station 25+00 to 32+00.

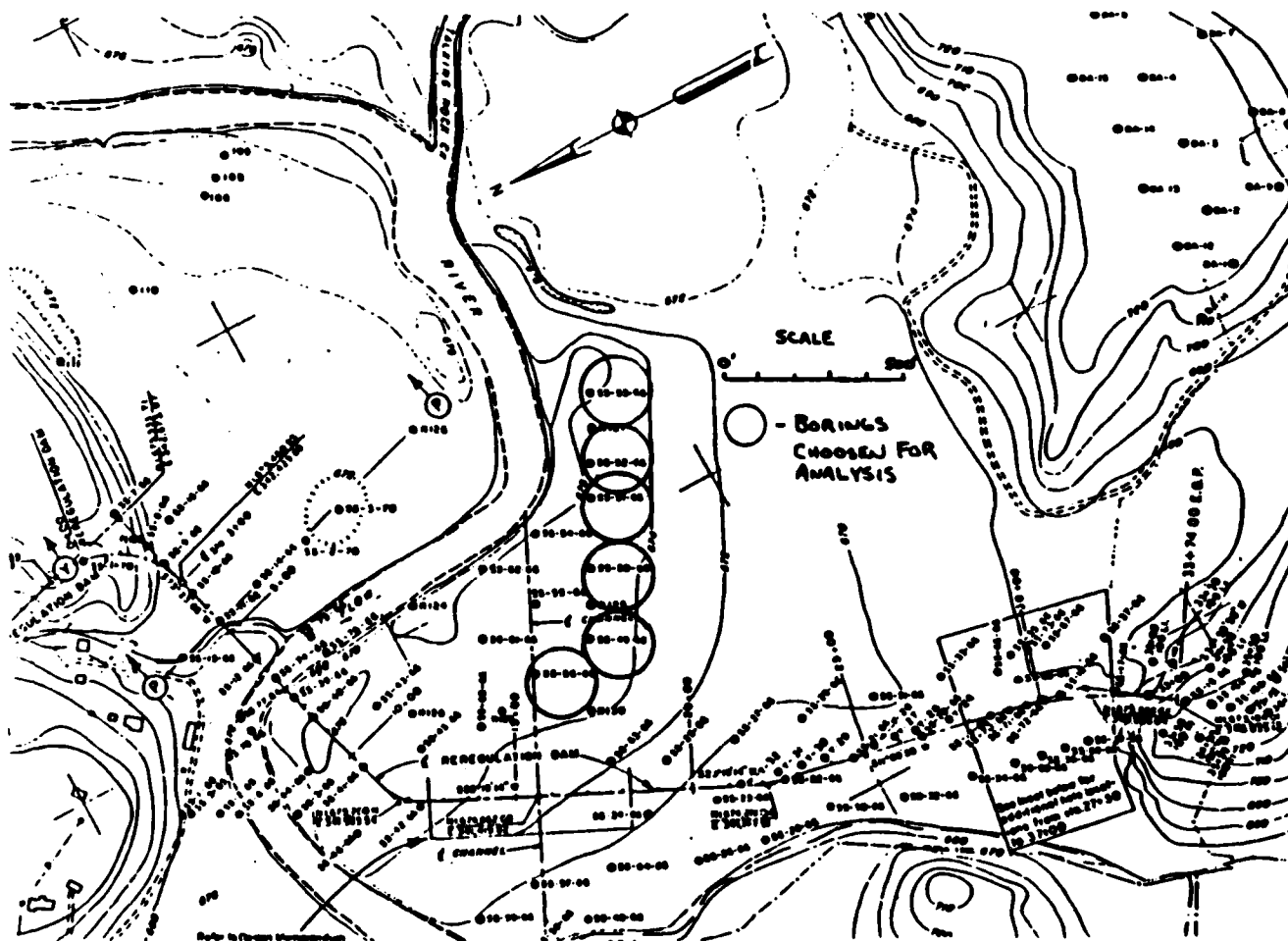


Figure 29a. Test Borings for Pattern Recognition.



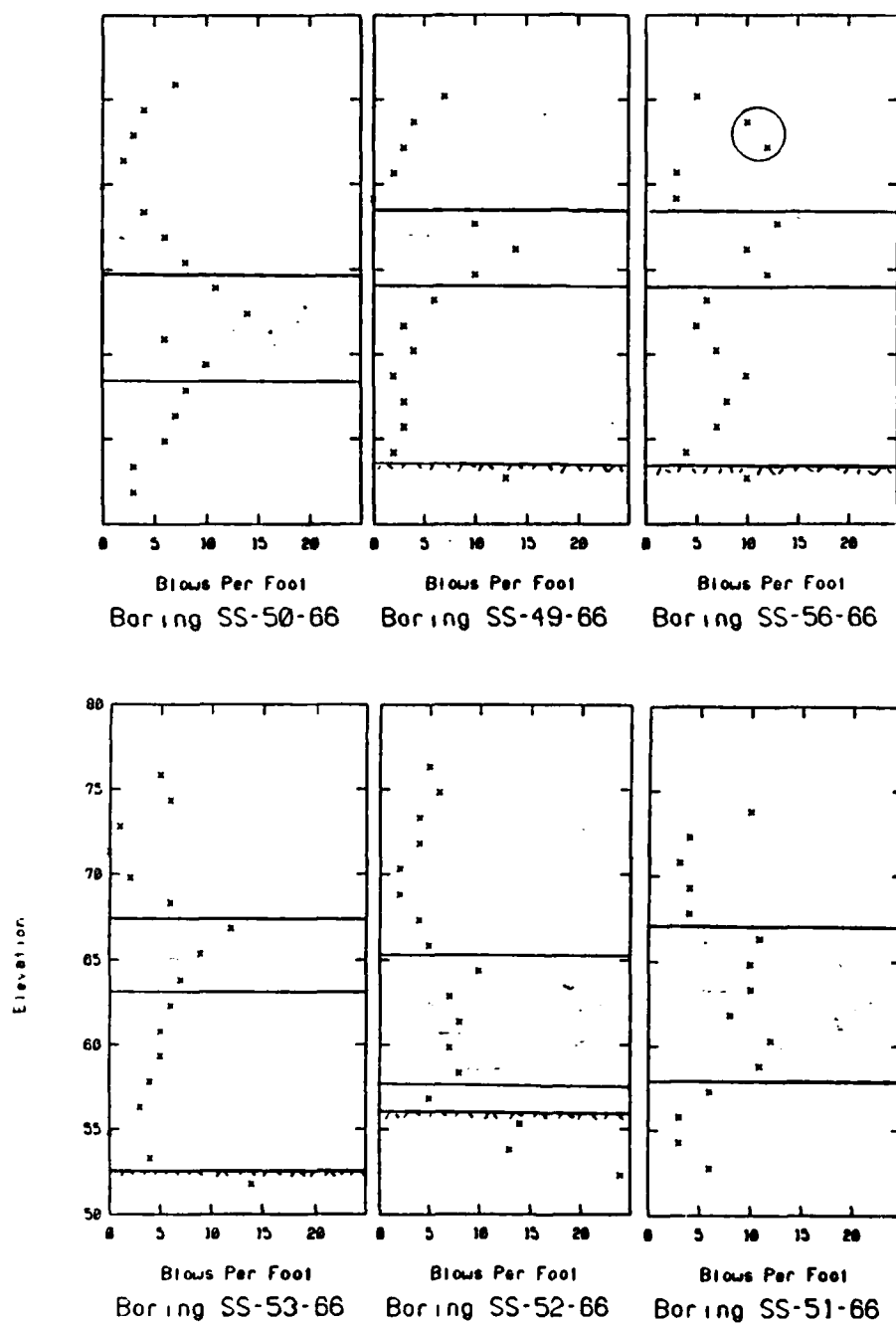


Figure 29c. Result of Pattern Recognition Experiment.

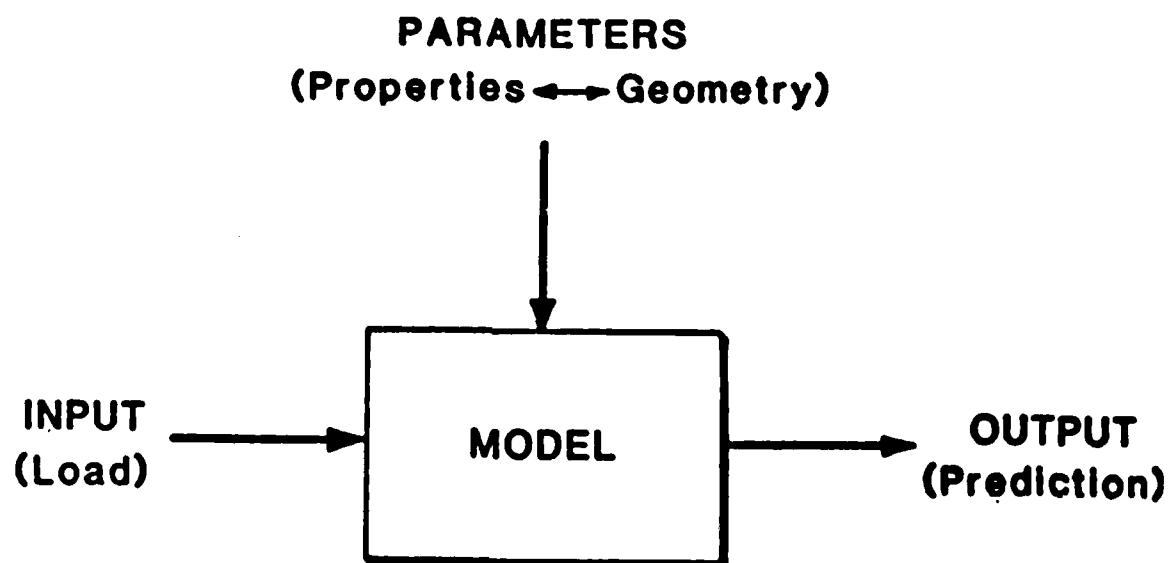


Figure 30. Data, Parameters, and Engineering Models.

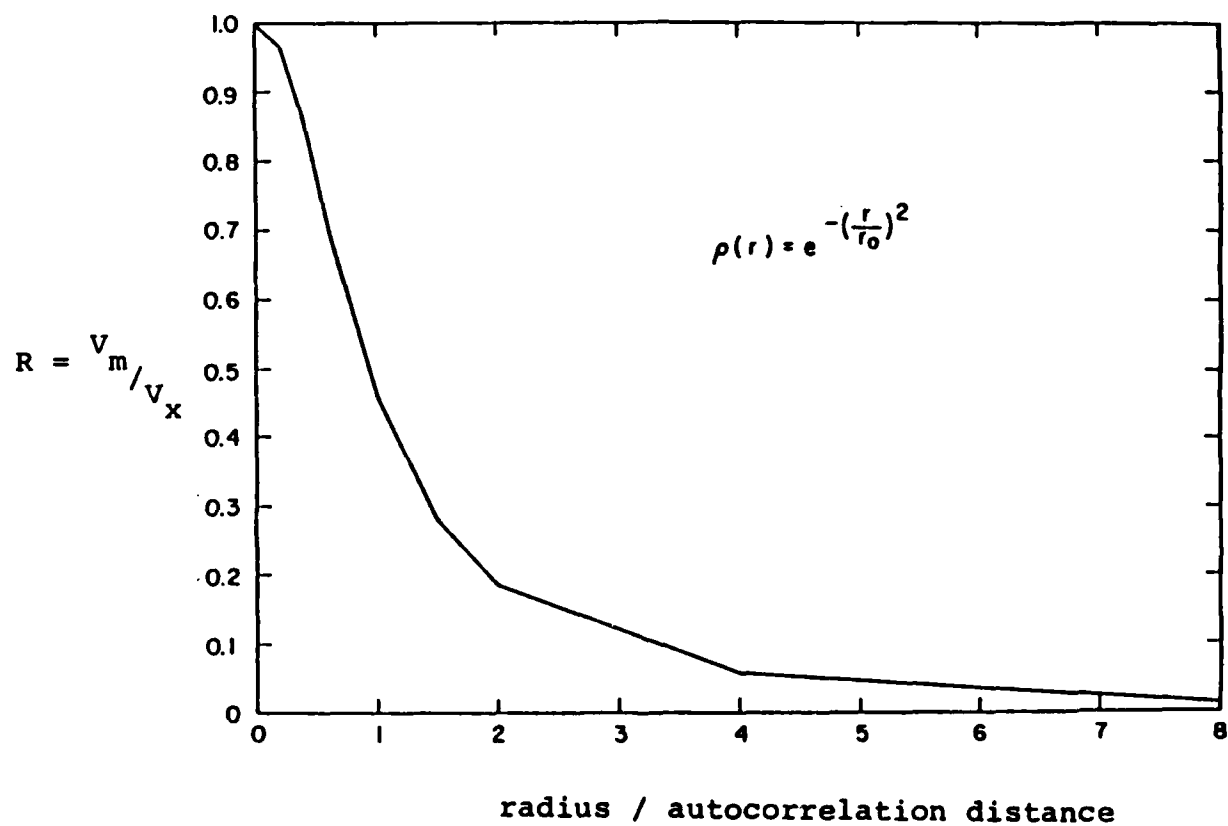


Figure 31. Size Effect Factor R for 1D Averaging, Exponential Autocorrelation Function, circular area.

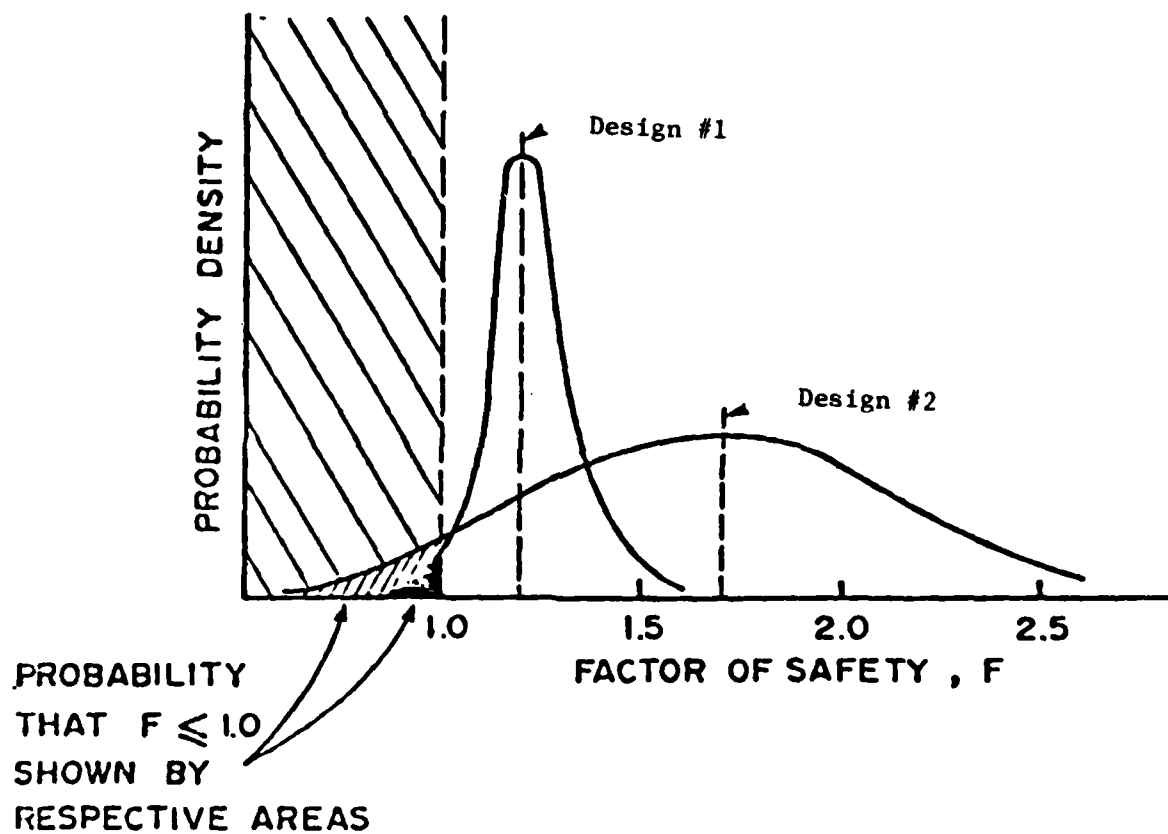


Figure 32. Comparison of Two Calculations Having Different Mean Factors of Safety and Different Standard Deviations.

TOTAL NUMBER OF DATA • 73
 MEAN • 31.699 STD DEV • 7.4538
 NUMBER OF OUTLIERS BEYOND LOWER LIMIT OF PLOT • 0
 NUMBER OF OUTLIERS BEYOND UPPER LIMIT OF PLOT • 0

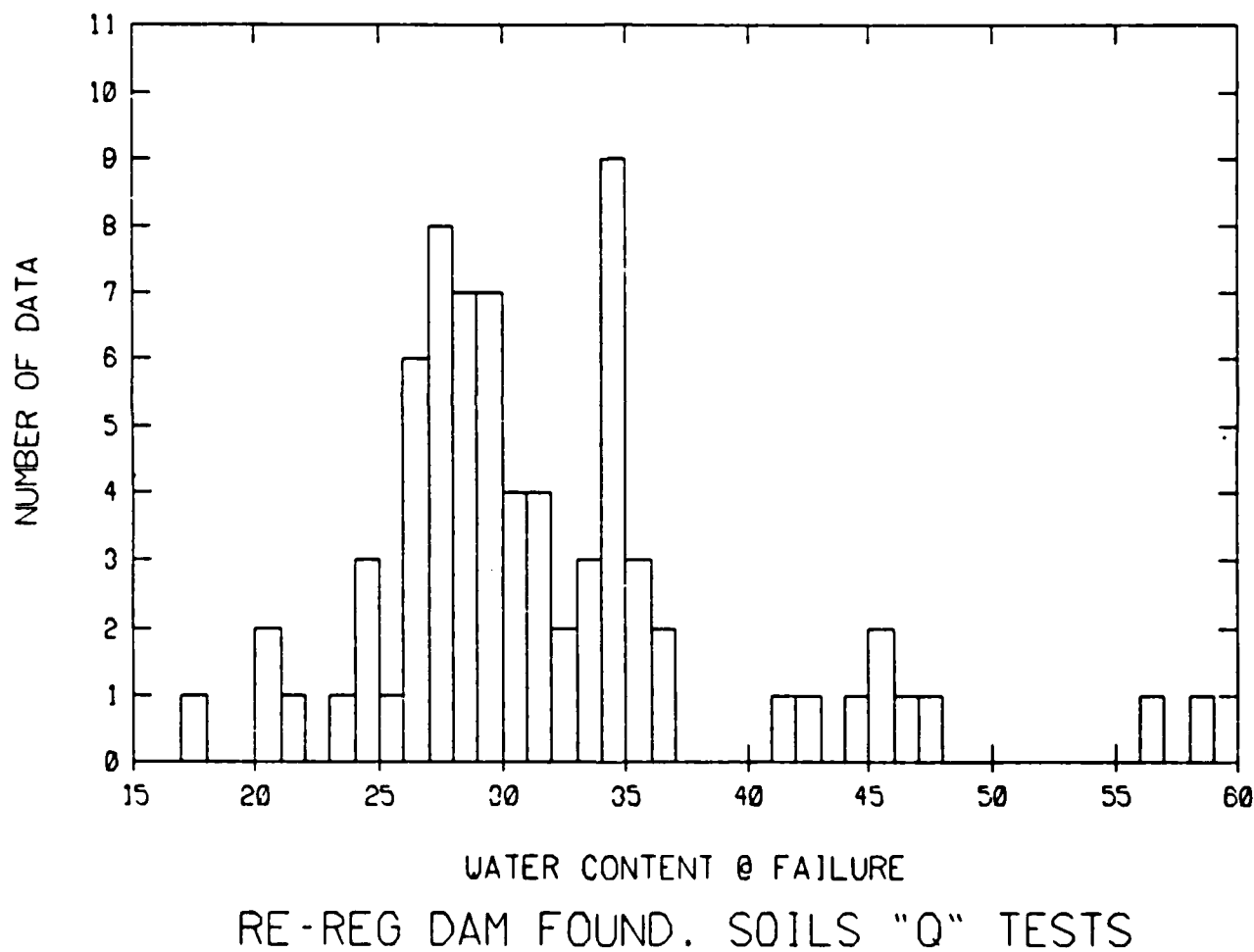


Figure 33a. Water Content at Failure: Q-Tests Reregulation
 Dam Foundation Soils.

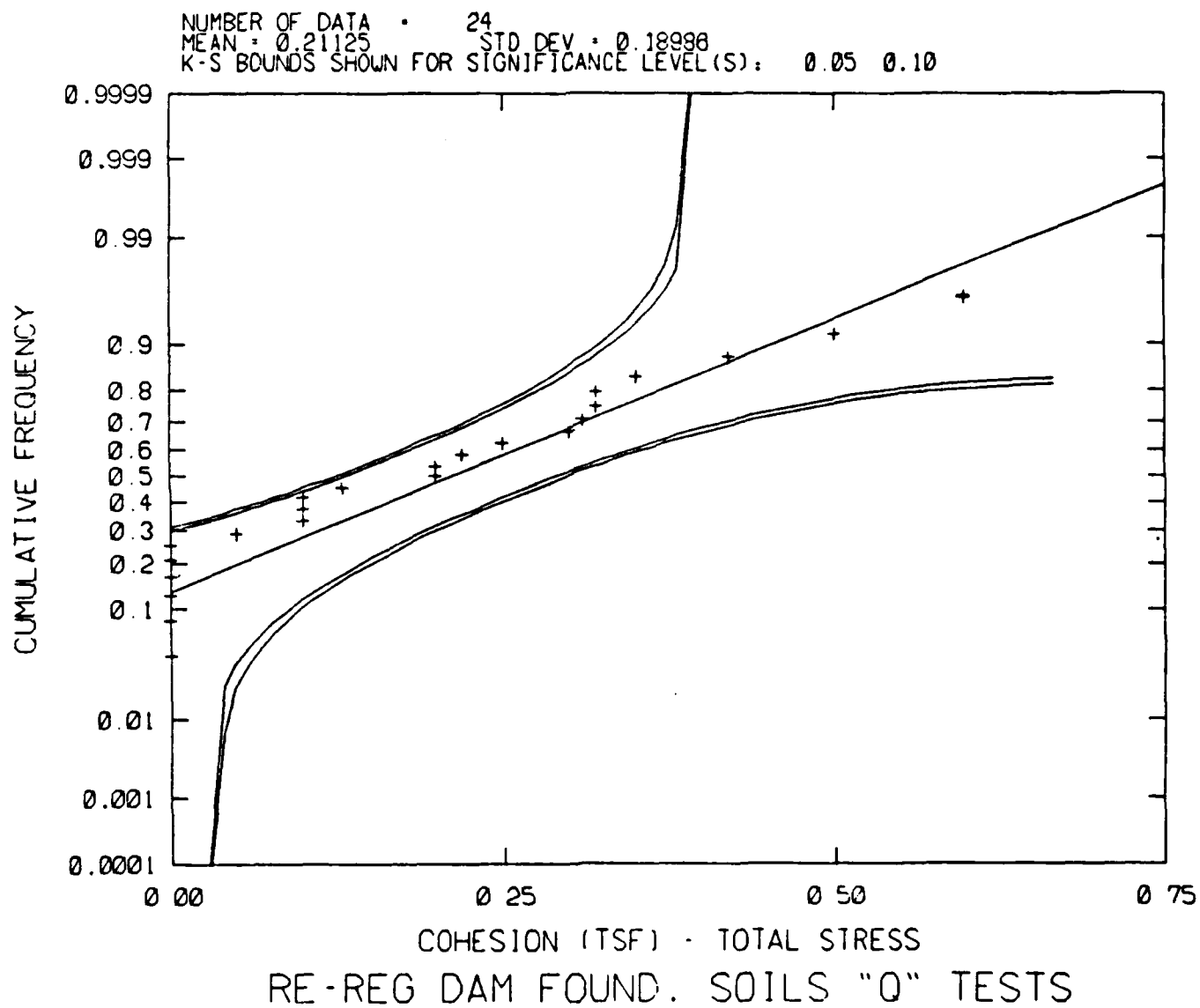


Figure 33b. Cohesion: Q-Tests Reregulation Dam Foundation Soils.

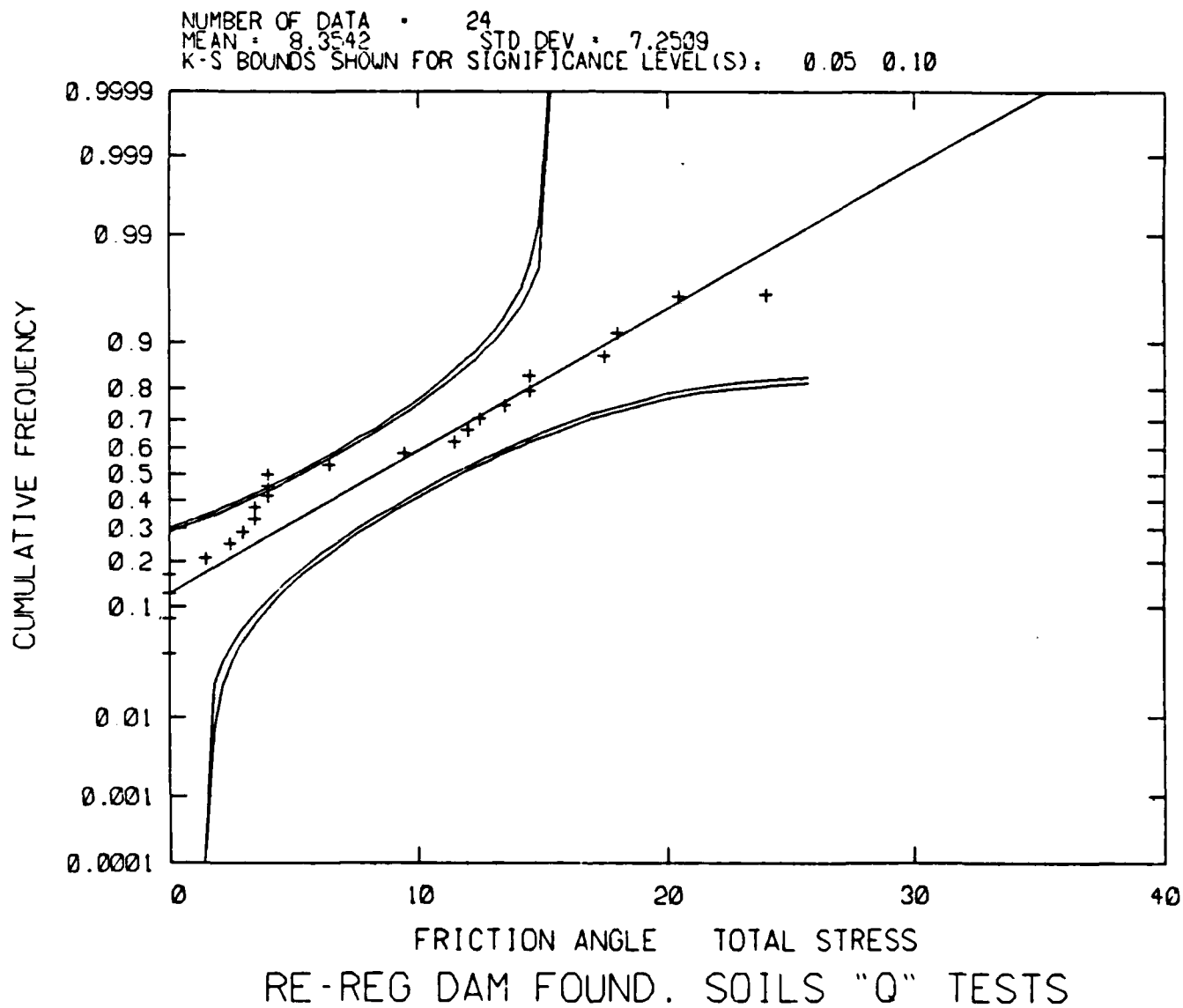


Figure 33c. Friction Angle: Q-Tests Reregulation Dam Foundation Soils.

TOTAL NUMBER OF DATA · 73
 MEAN · 32.581 STD DEV · 6.5427
 NUMBER OF OUTLIERS BEYOND LOWER LIMIT OF PLOT · 0
 NUMBER OF OUTLIERS BEYOND UPPER LIMIT OF PLOT · 0

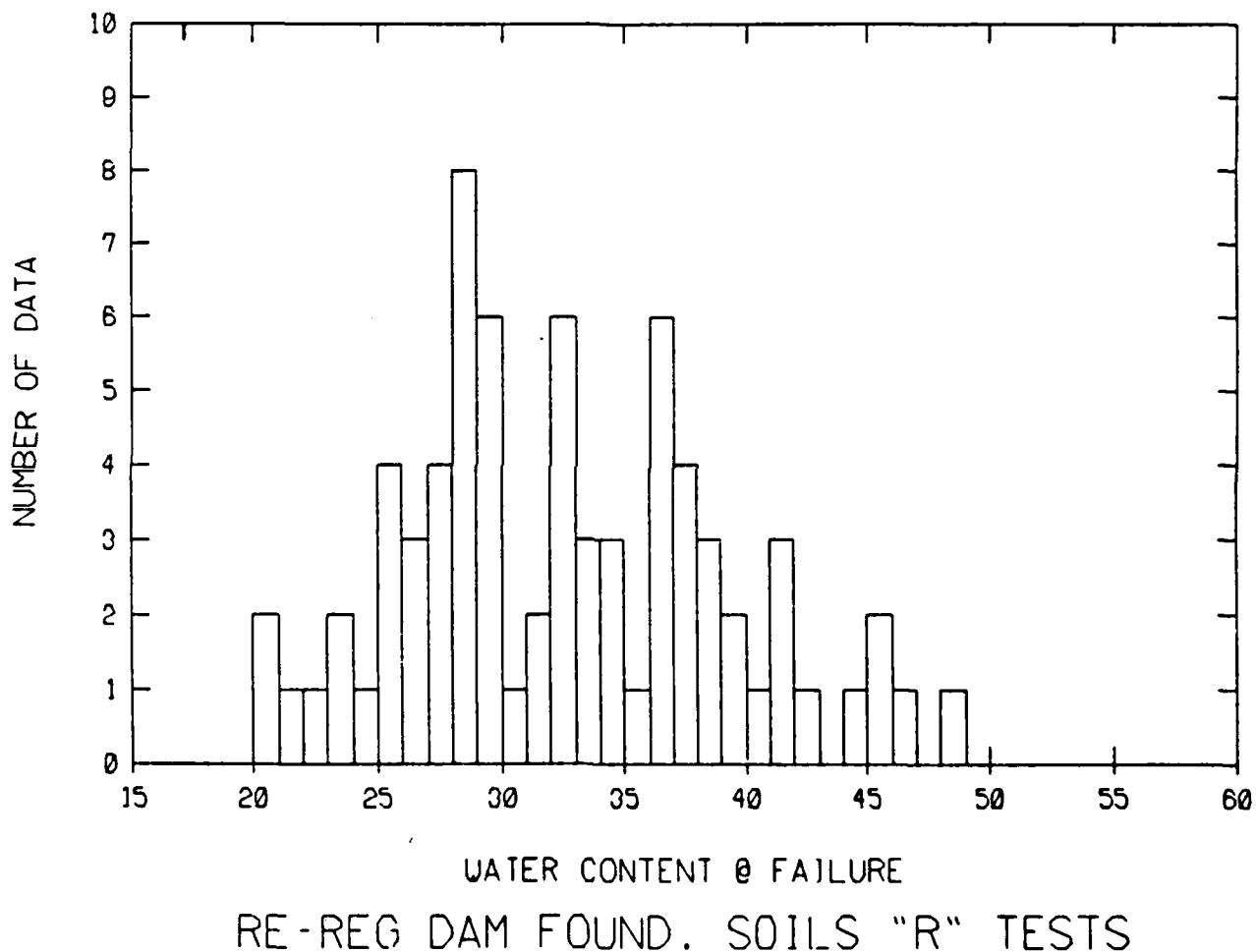


Figure 34a. Water Content at Failure: R-Tests Reregulation
 Dam Foundation Soils.

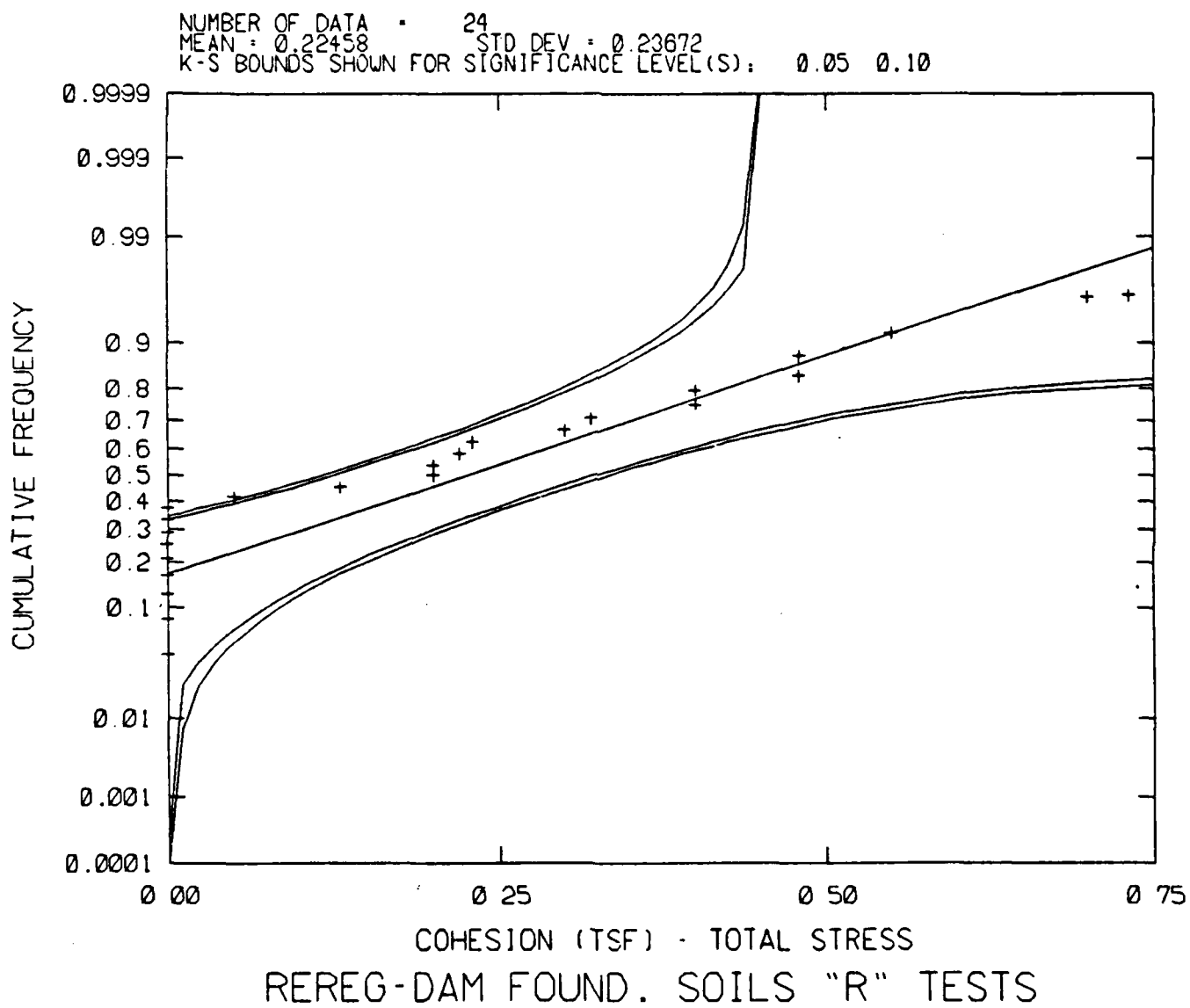


Figure 34b. Cohesion: R-Tests Reregulation Dam Foundation Soils.

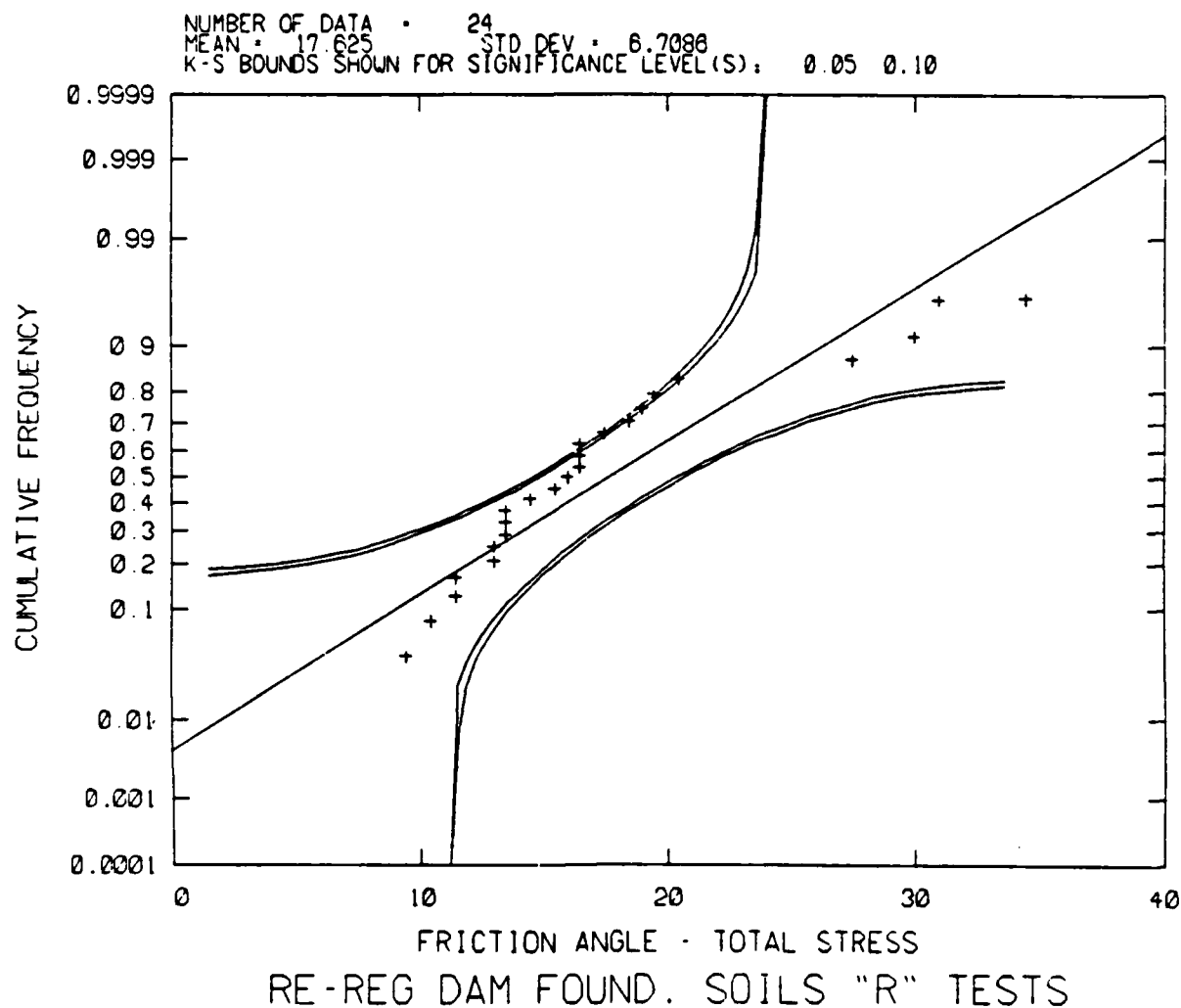


Figure 34c. Friction Angle: R-Tests Reregulation Dam Foundation Soils.

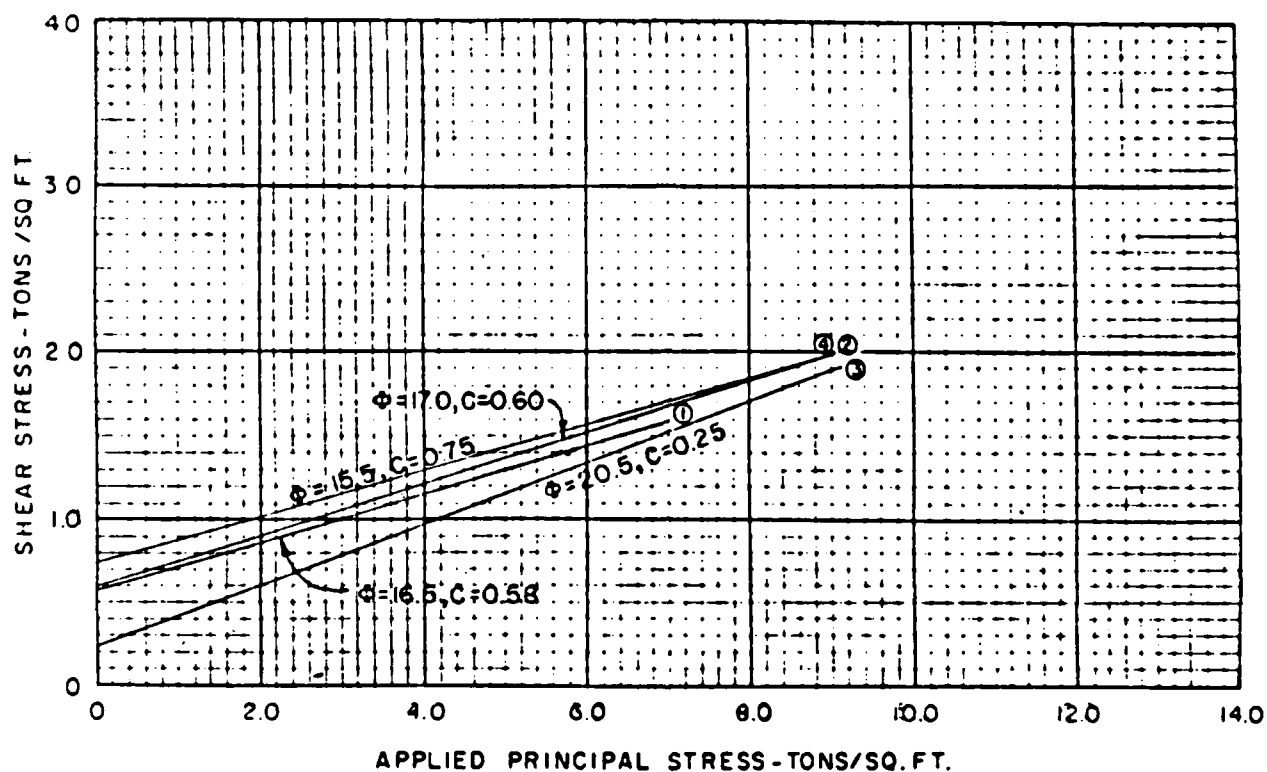


Figure 35. Mohr Diagram for Strength Tests on Borrow Material.

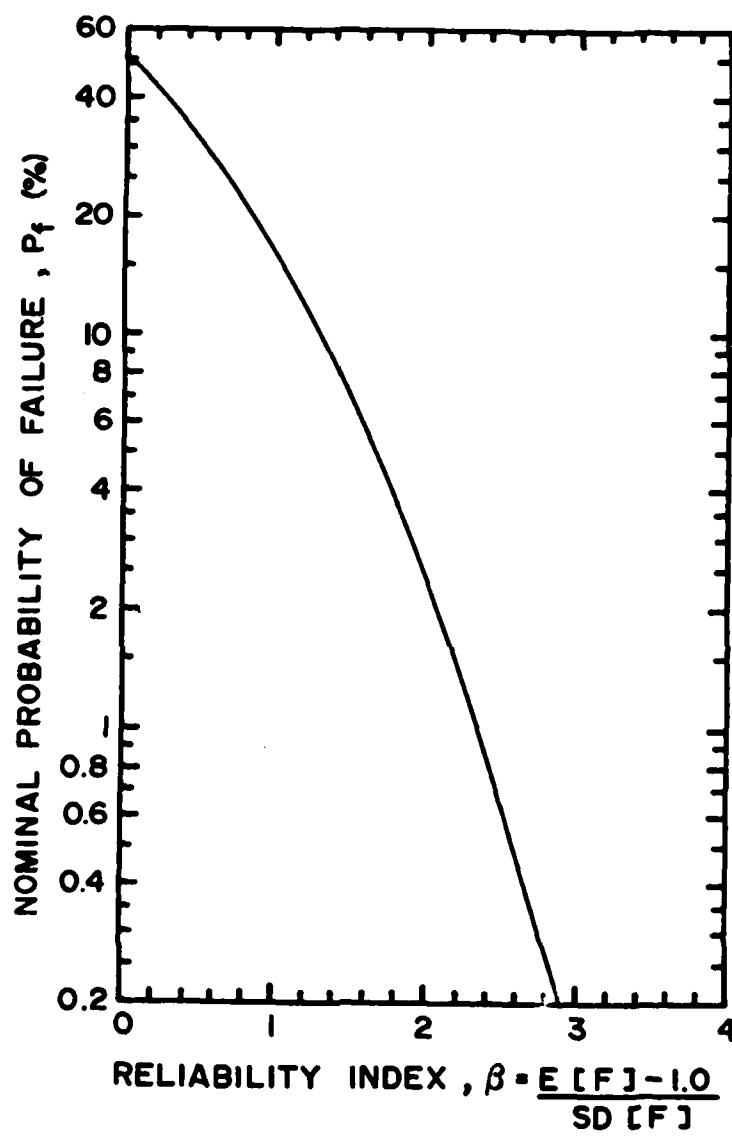
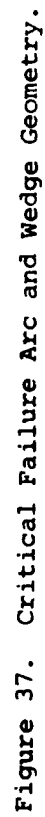


Figure 36. Relation of Nominal Probability of Failure to Reliability Index for Normal Distribution.



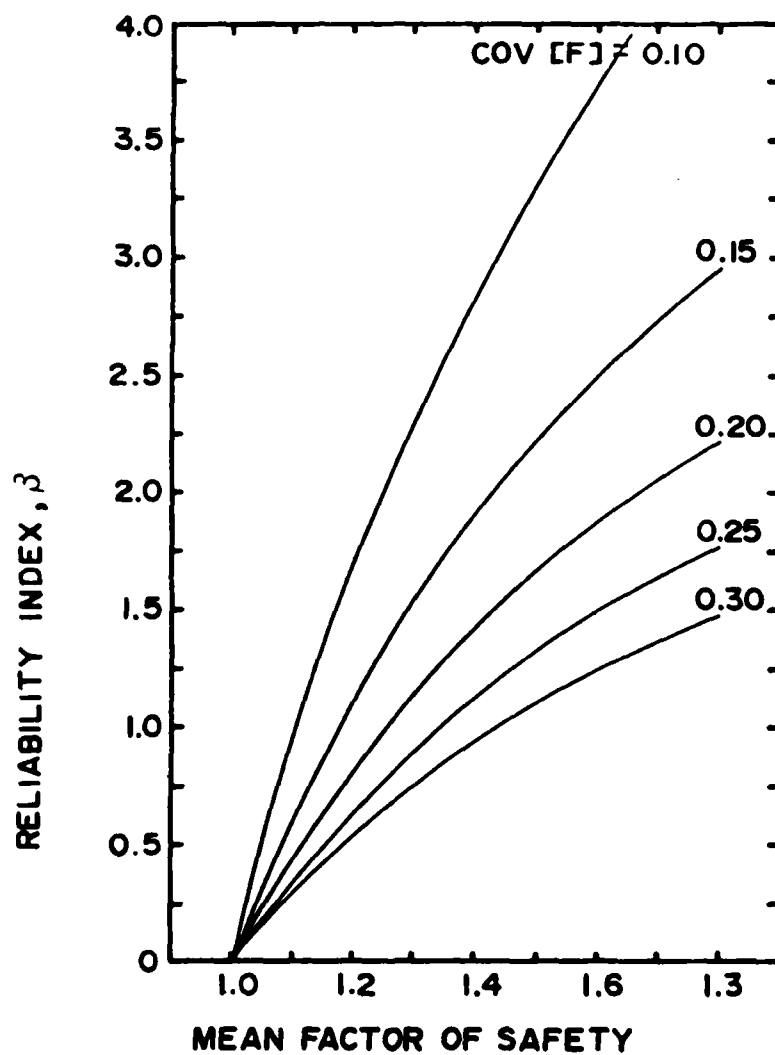


Figure 38. Relation Between Factor of Safety, Coefficient of Variation, and Reliability Index.

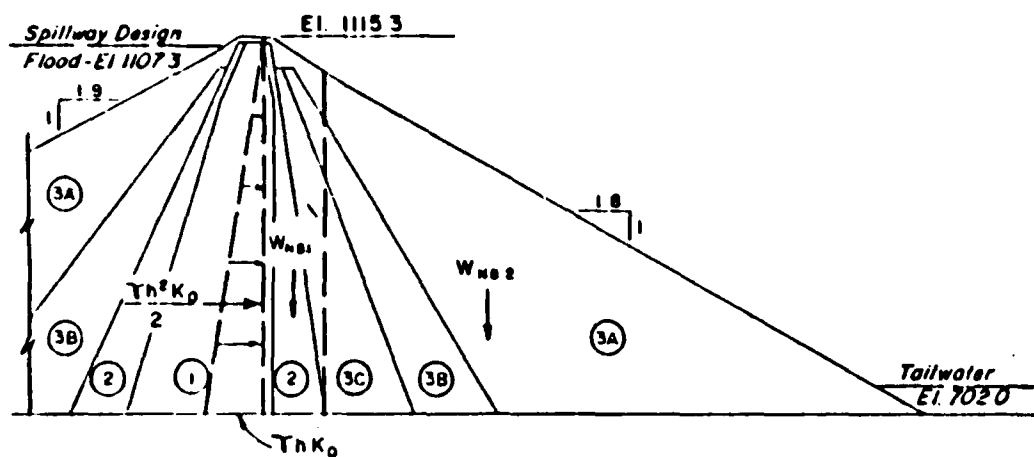


Figure 39. Critical Wedge for Stability Analysis of Main Embankment.

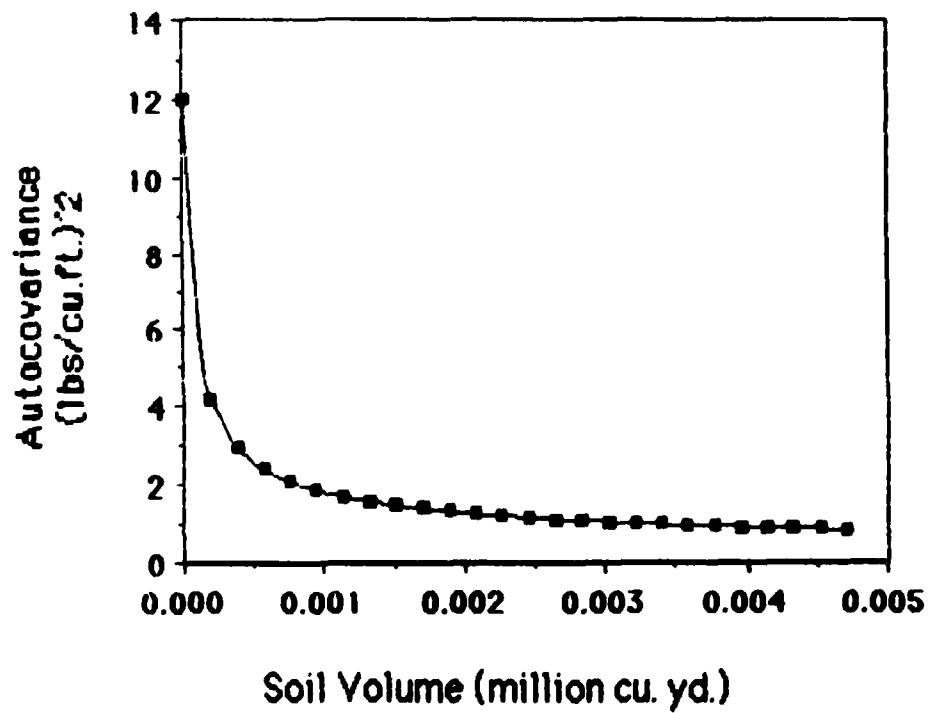


Figure 40. Estimate of the Variance of Average Compacted Dry Density as a Function of Fill Volume.

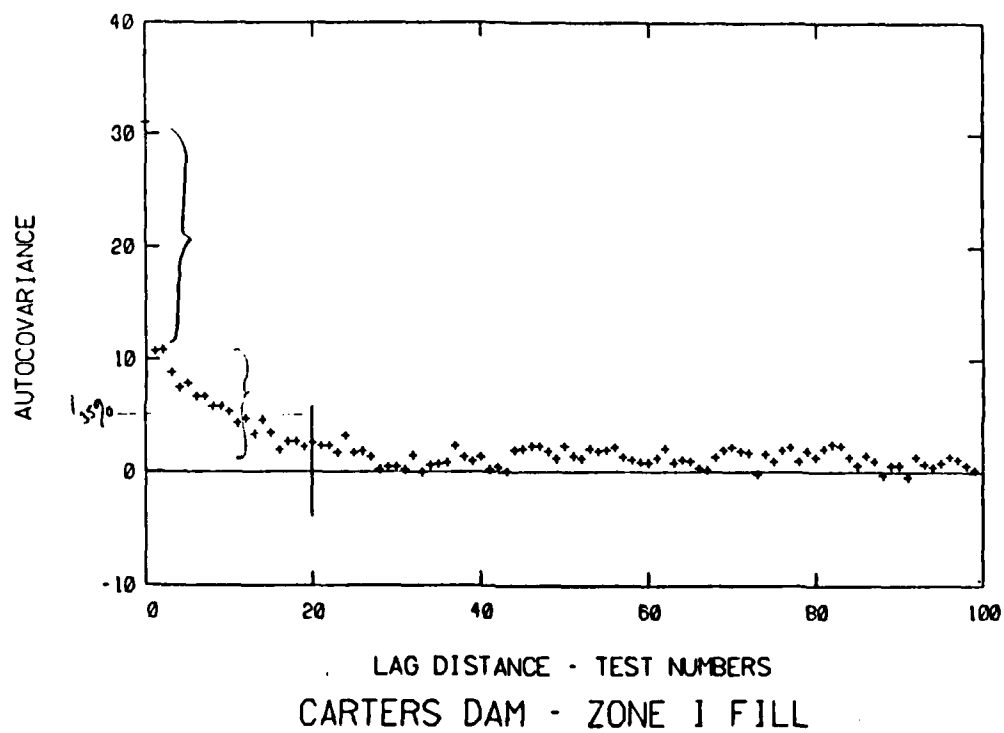


Figure 41. Autocovariance Function of Compacted Dry Density.

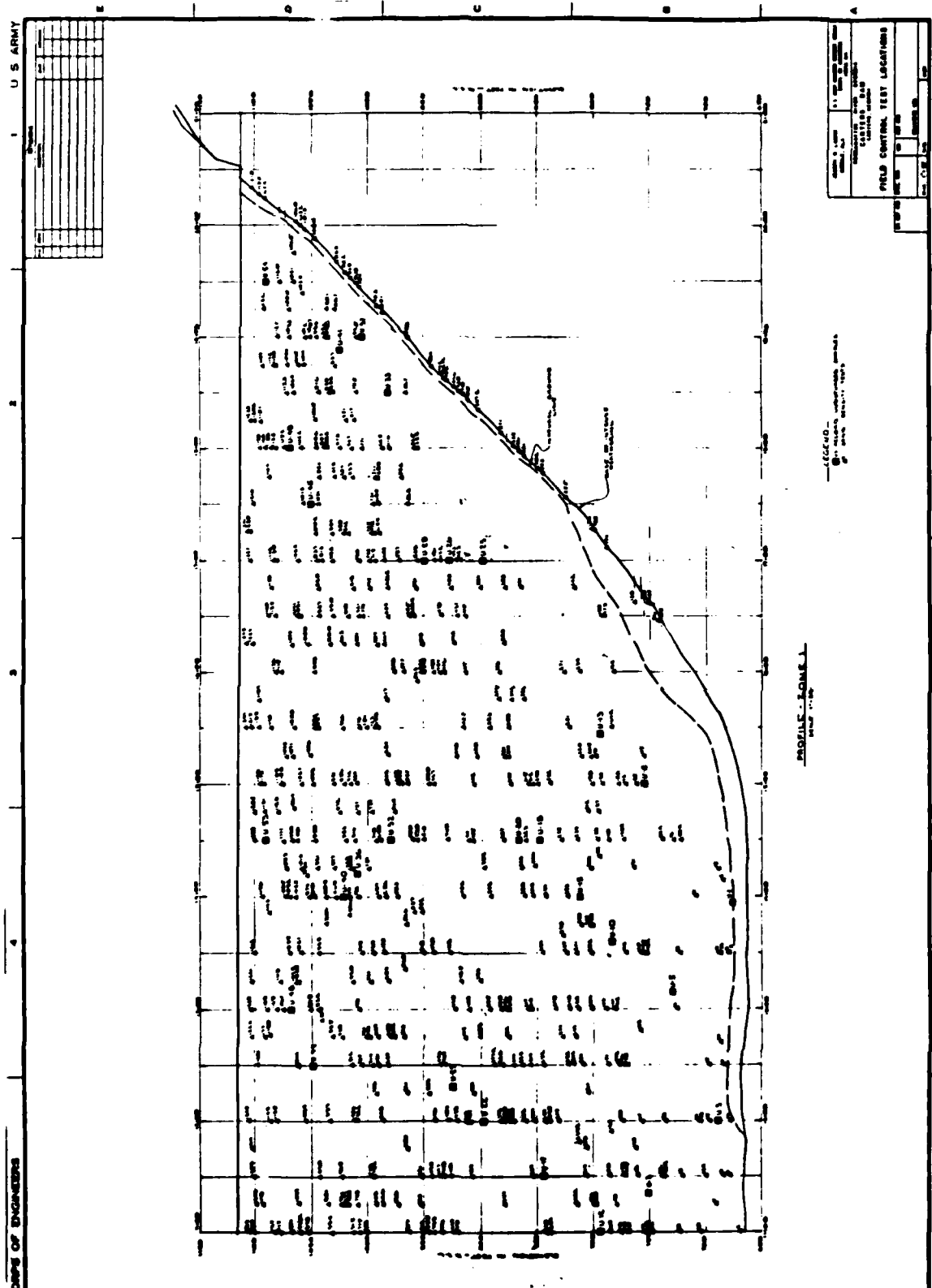


Figure 42a. Location of Compaction Control Tests in Zone 1, RHS.

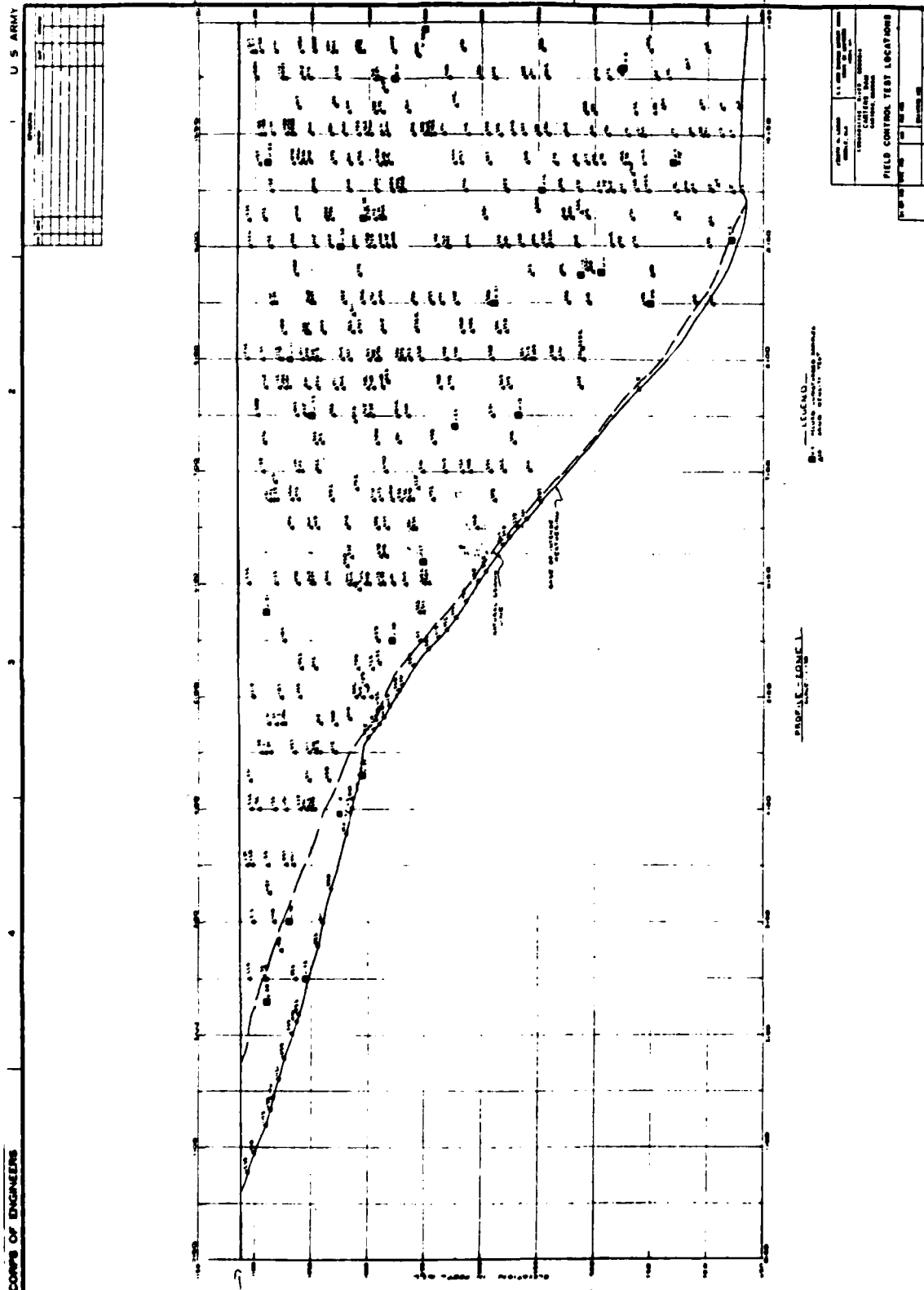


Figure 42b. Location of Compaction Control Tests in Zone 1, LHS.

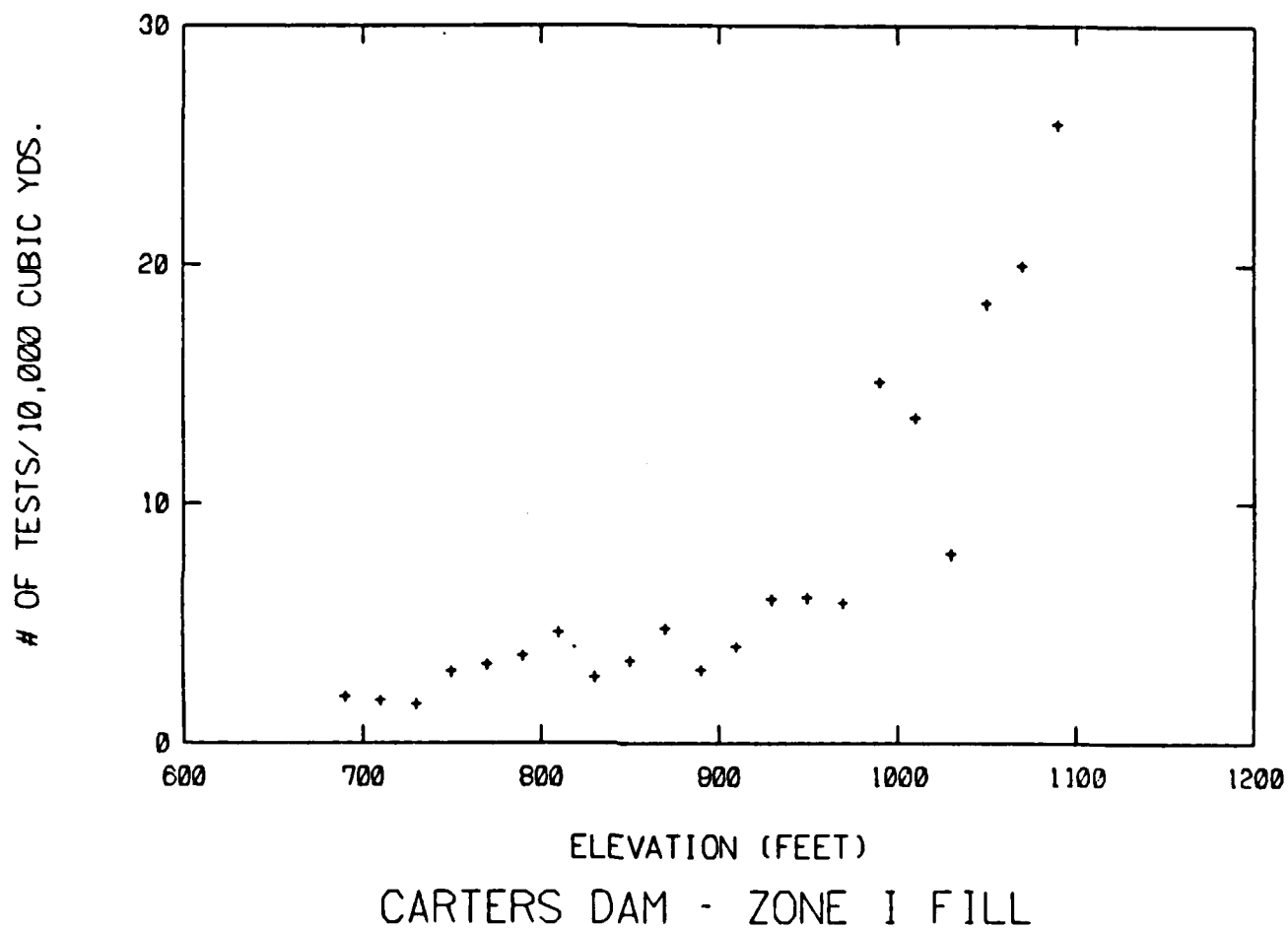
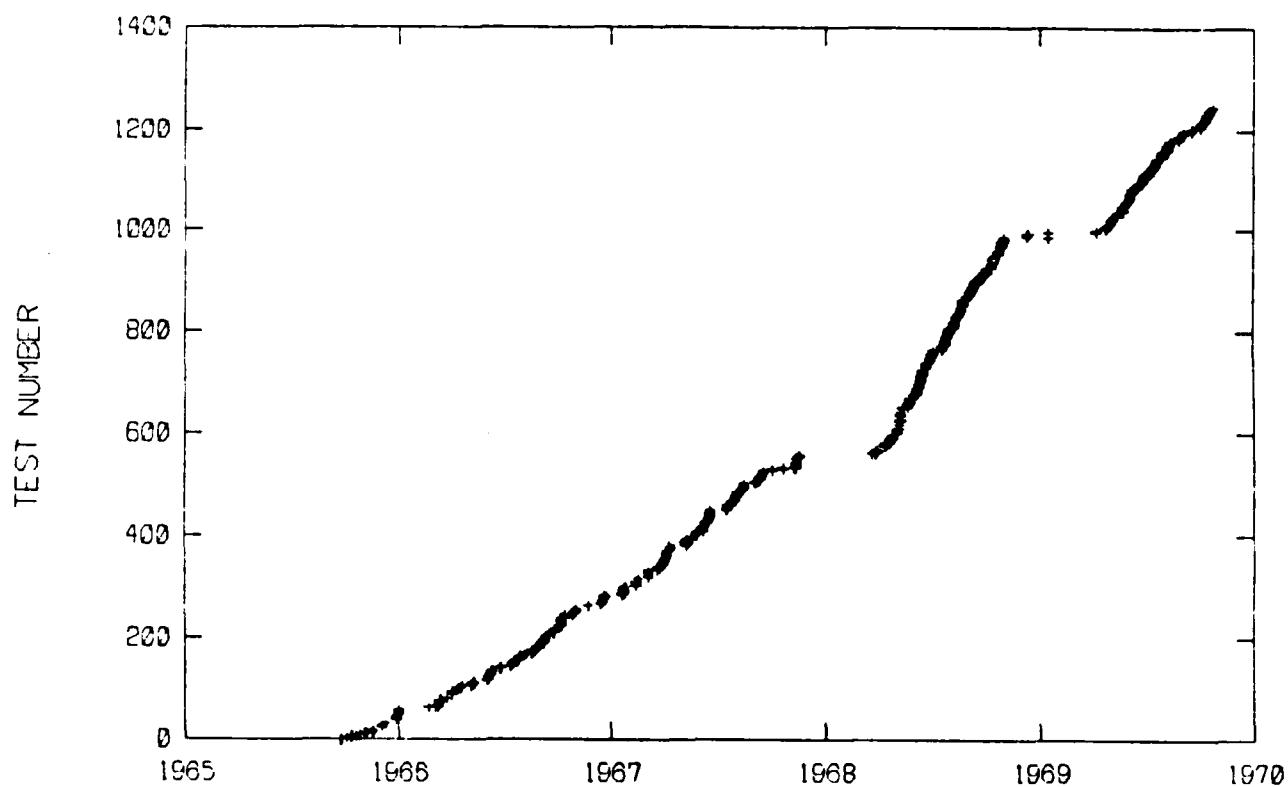


Figure 43. Number of Control Tests Per Volume of Fill, Zone 1.



DATE OF DENSITY TEST (YEARS)
CARTERS DAM - ZONE I FILL

Figure 44. Date of Control Tests, Zone 1.

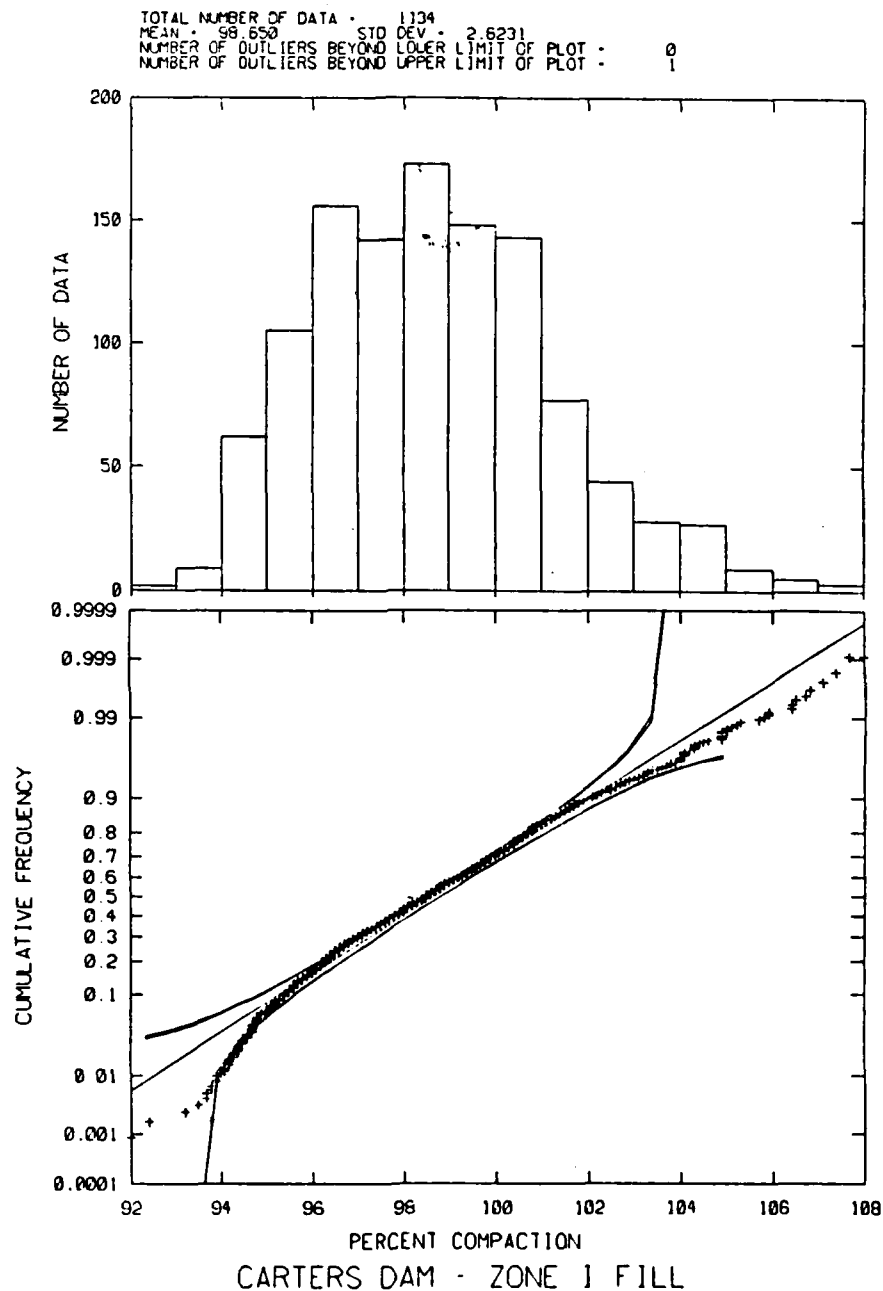


Figure 45. Histograms and Probability Plots of Percent Compaction Data from Zone 1.

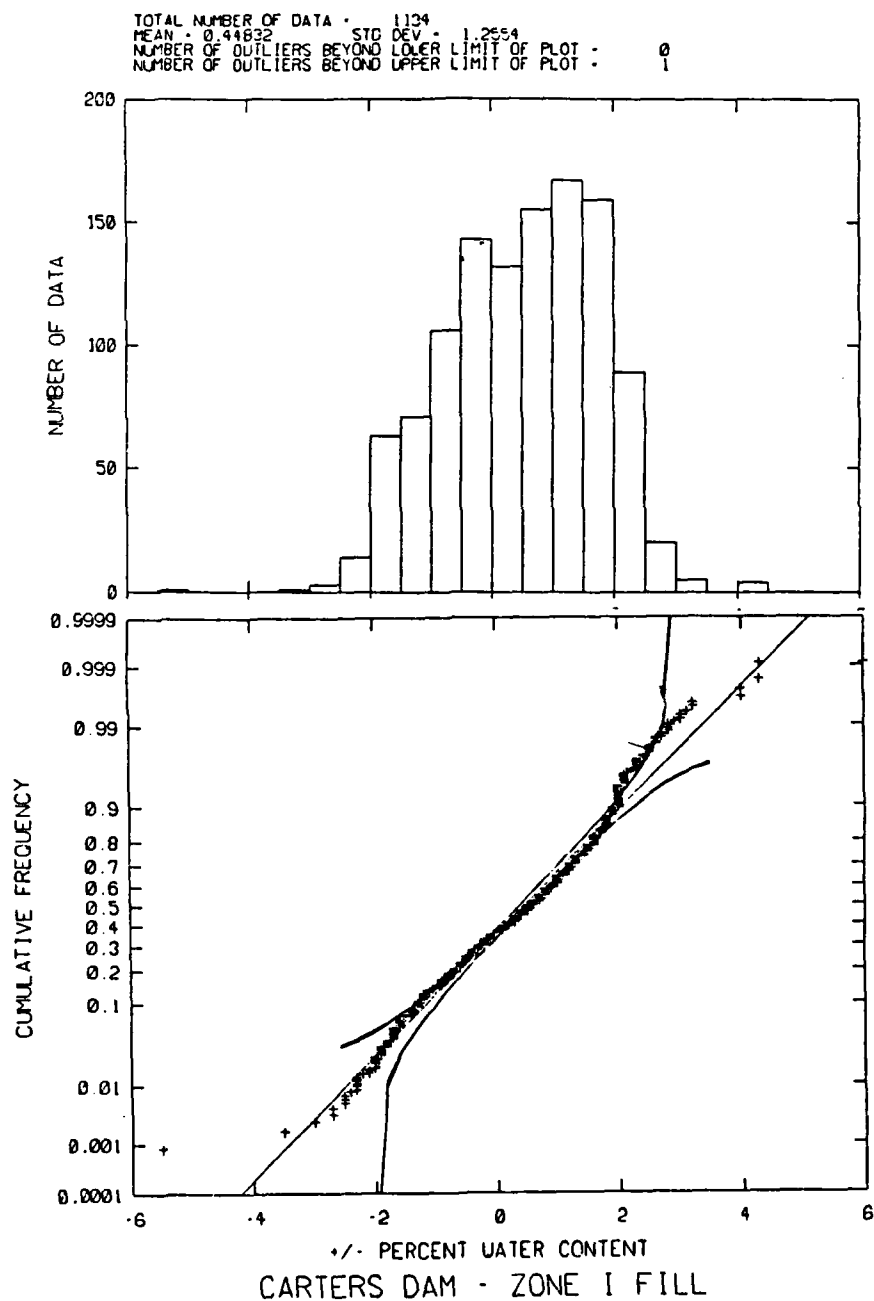


Figure 46. Histograms and Probability Plots of +/- Percent Water Content Data from Zone 1.

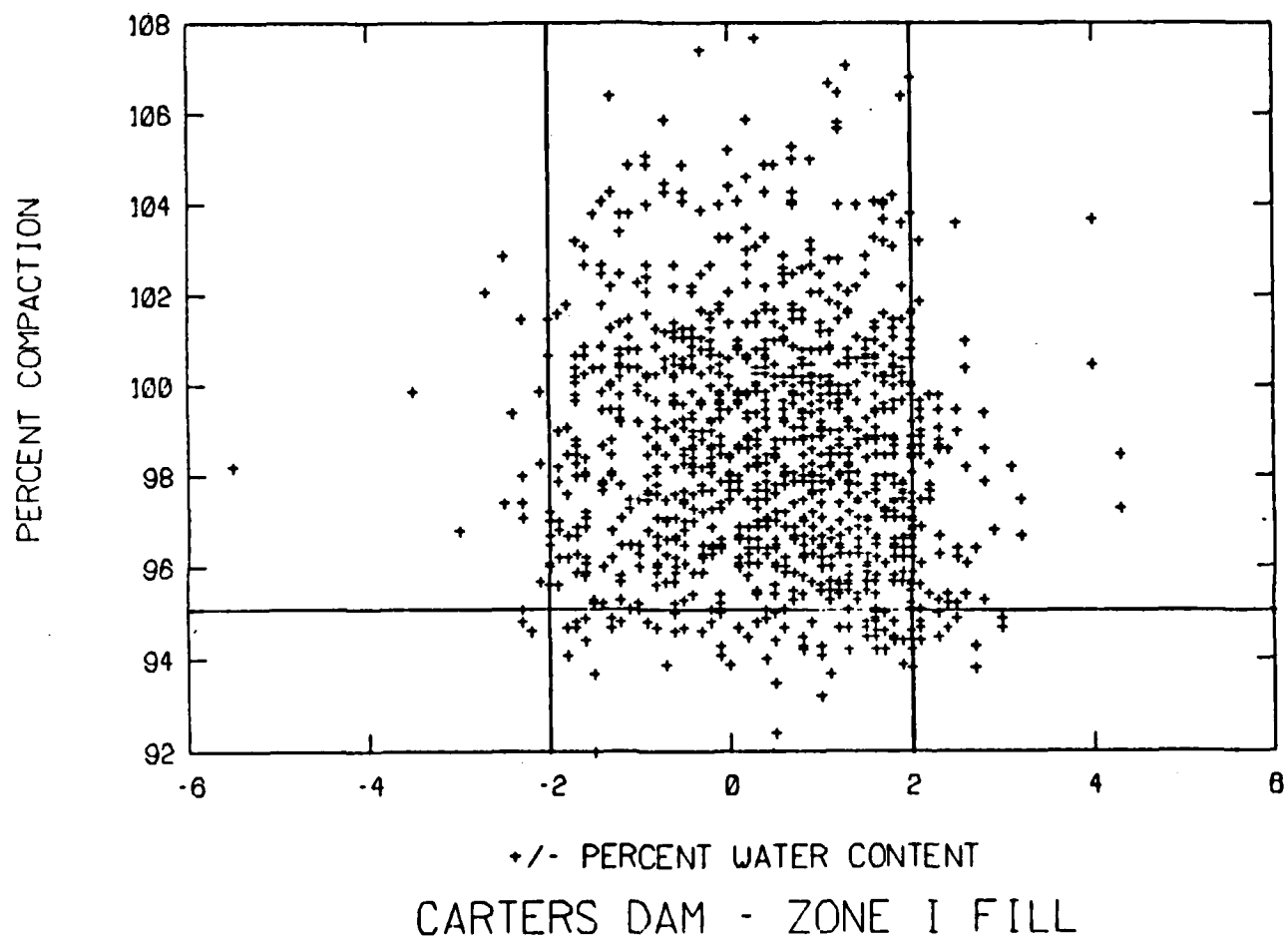


Figure 47. Percent Compaction vs. Water Content.

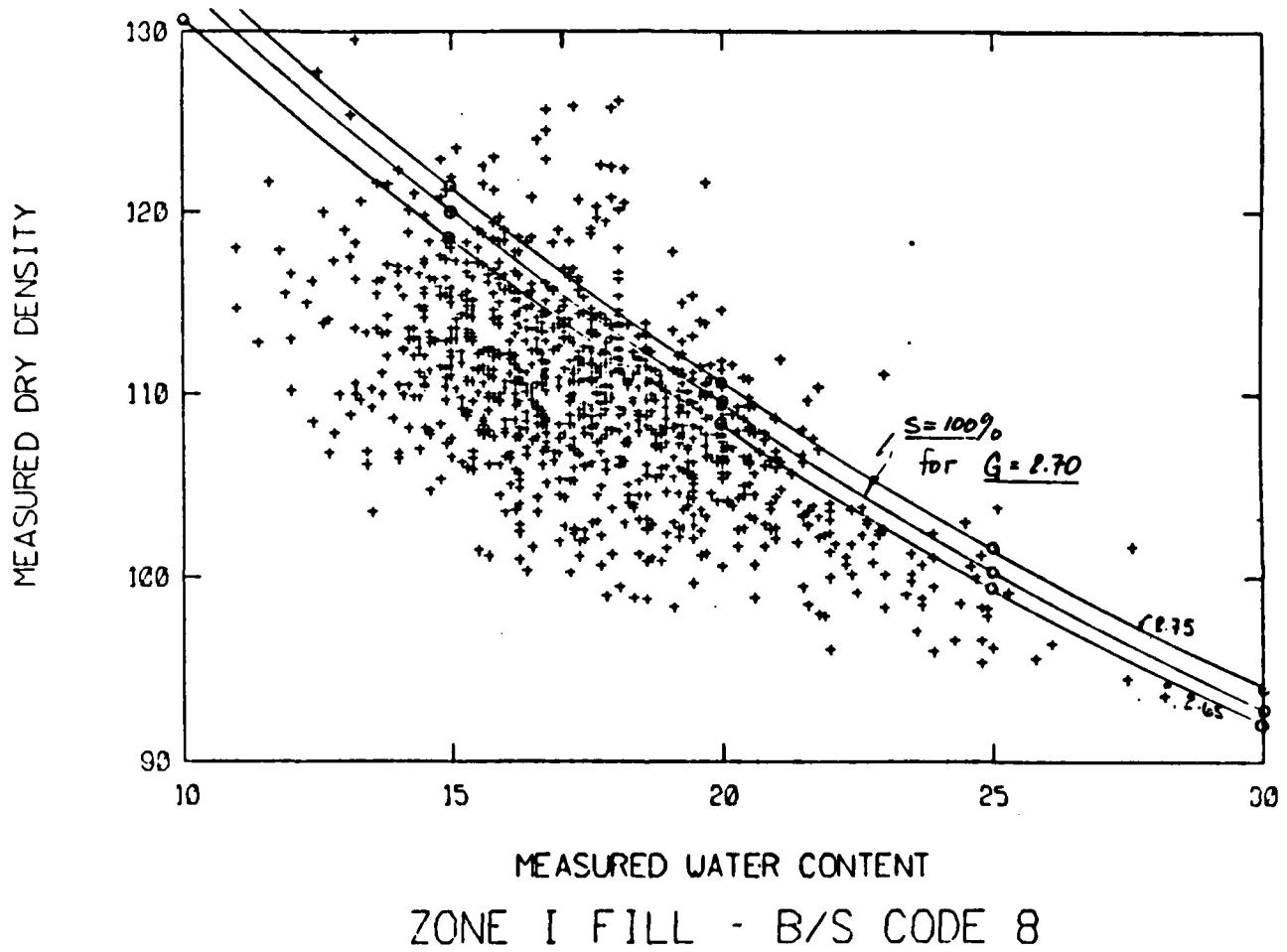


Figure 48. Compaction Curve, Water Content vs. Compacted Dry Density.

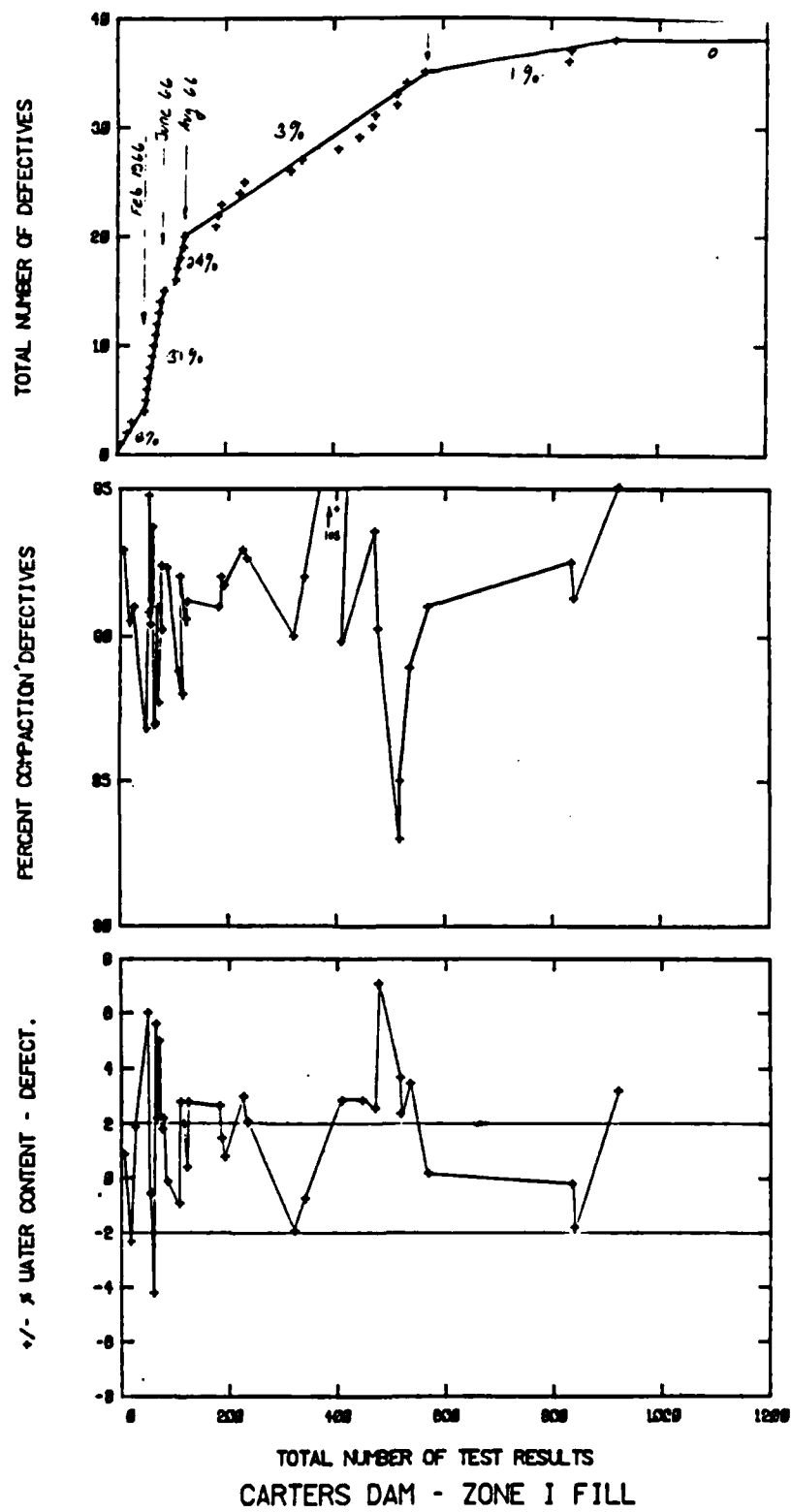


Figure 49. Rejection Chart of Density Tests.

TOTAL NUMBER OF DATA • 66
 MEAN • 98.745 STD DEV • 3.3714
 NUMBER OF OUTLIERS BEYOND LOWER LIMIT OF PLOT : 0
 NUMBER OF OUTLIERS BEYOND UPPER LIMIT OF PLOT : 0

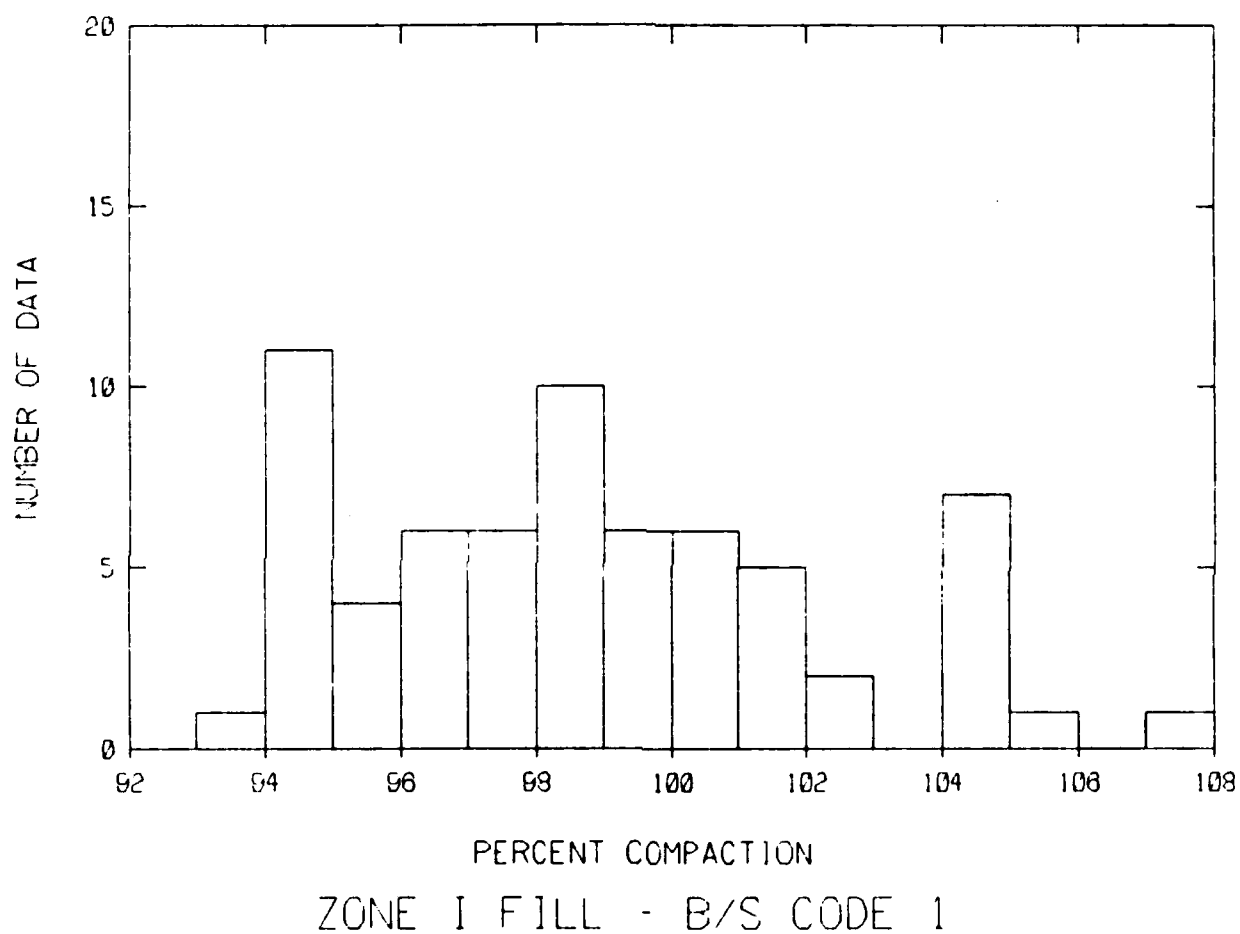


Figure 50. Histogram of Percent Compaction Data from Powerhouse Borrow Area.

TOTAL NUMBER OF DATA · 974
 MEAN · 98.717 STD DEV · 2.5599
 NUMBER OF OUTLIERS BEYOND LOWER LIMIT OF PLOT : 0
 NUMBER OF OUTLIERS BEYOND UPPER LIMIT OF PLOT : 1

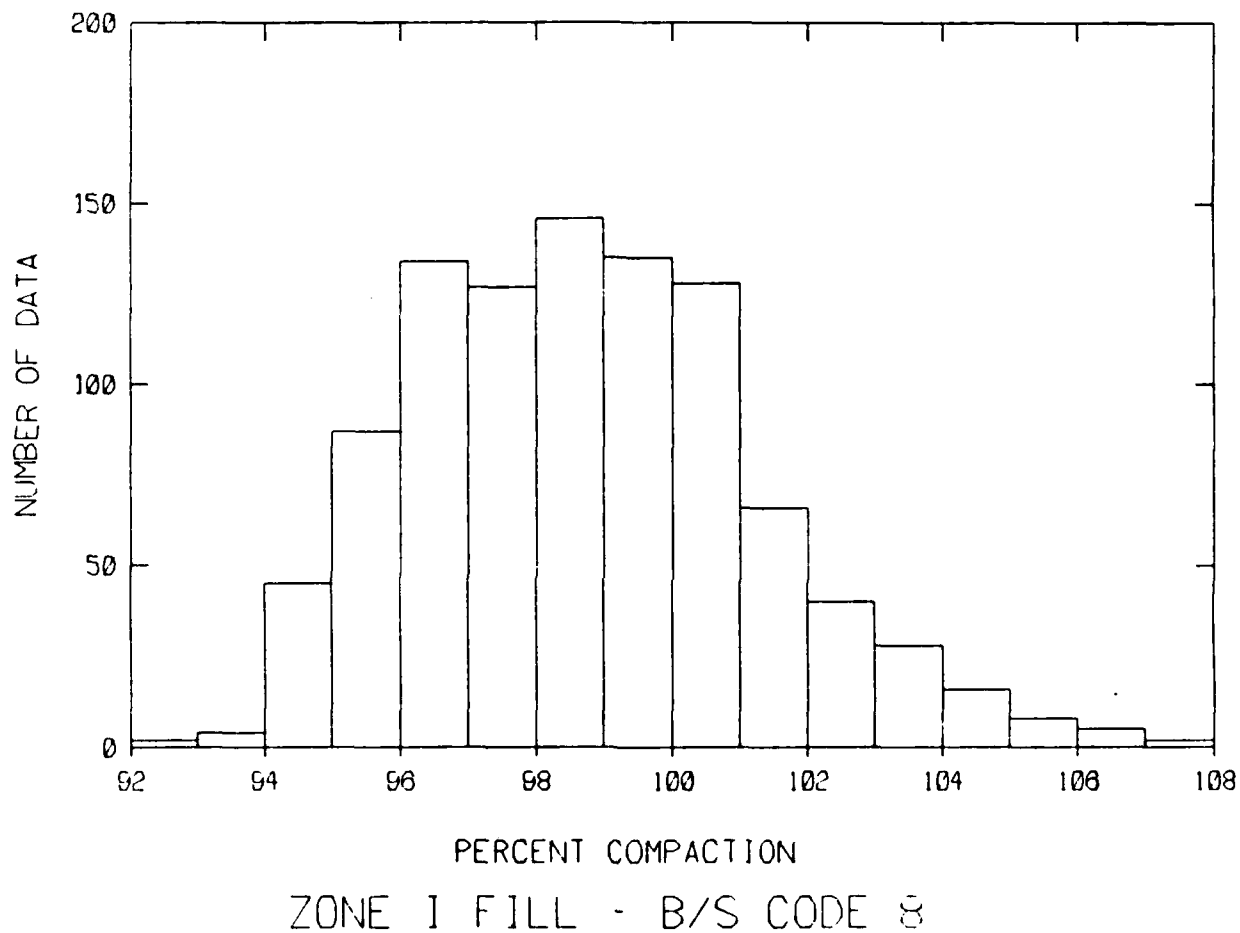


Figure 51. Histogram of Percent Compaction Data from Borrow Area 3.

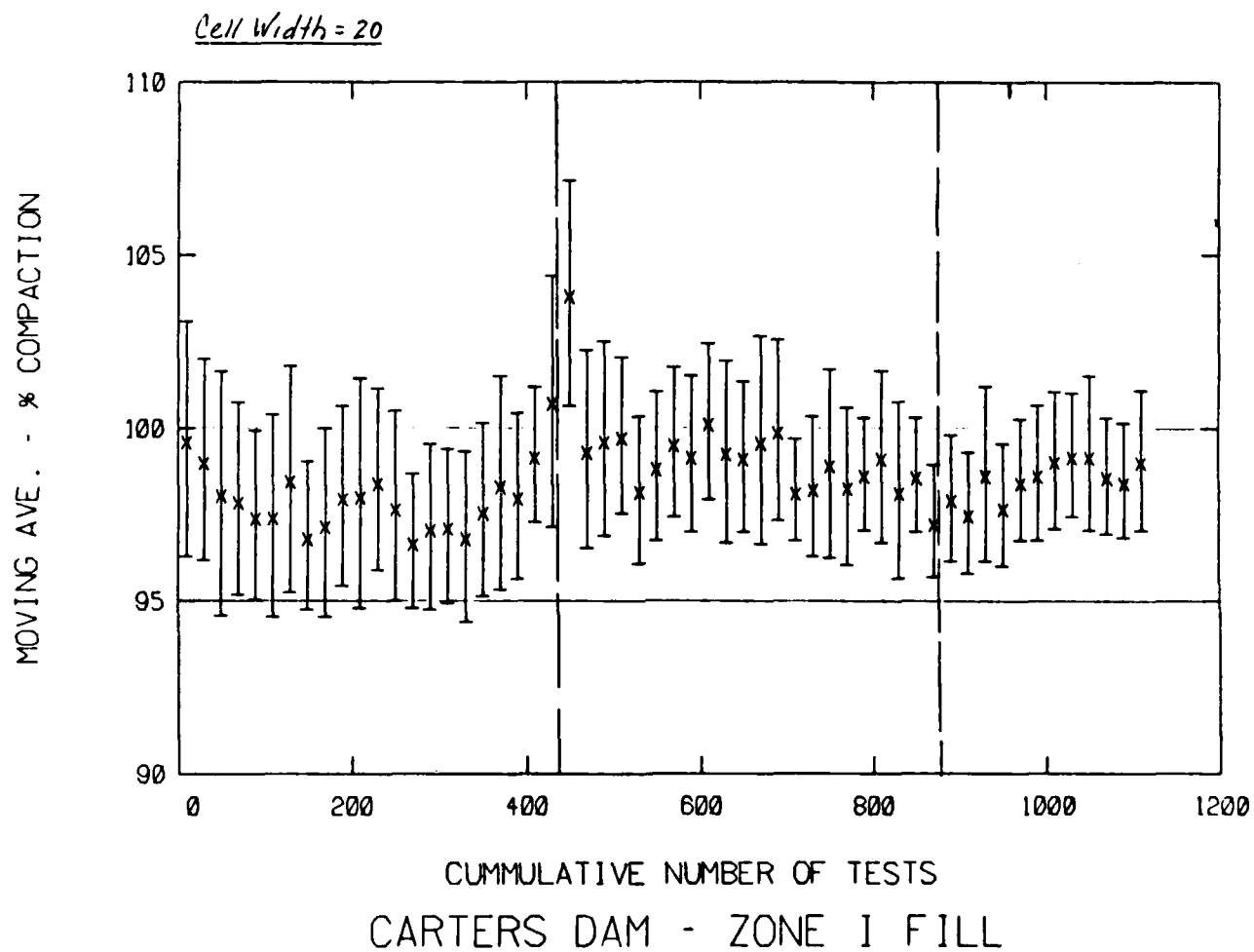


Figure 52. Moving Average Standard Deviation of Percent Compaction Control Data.

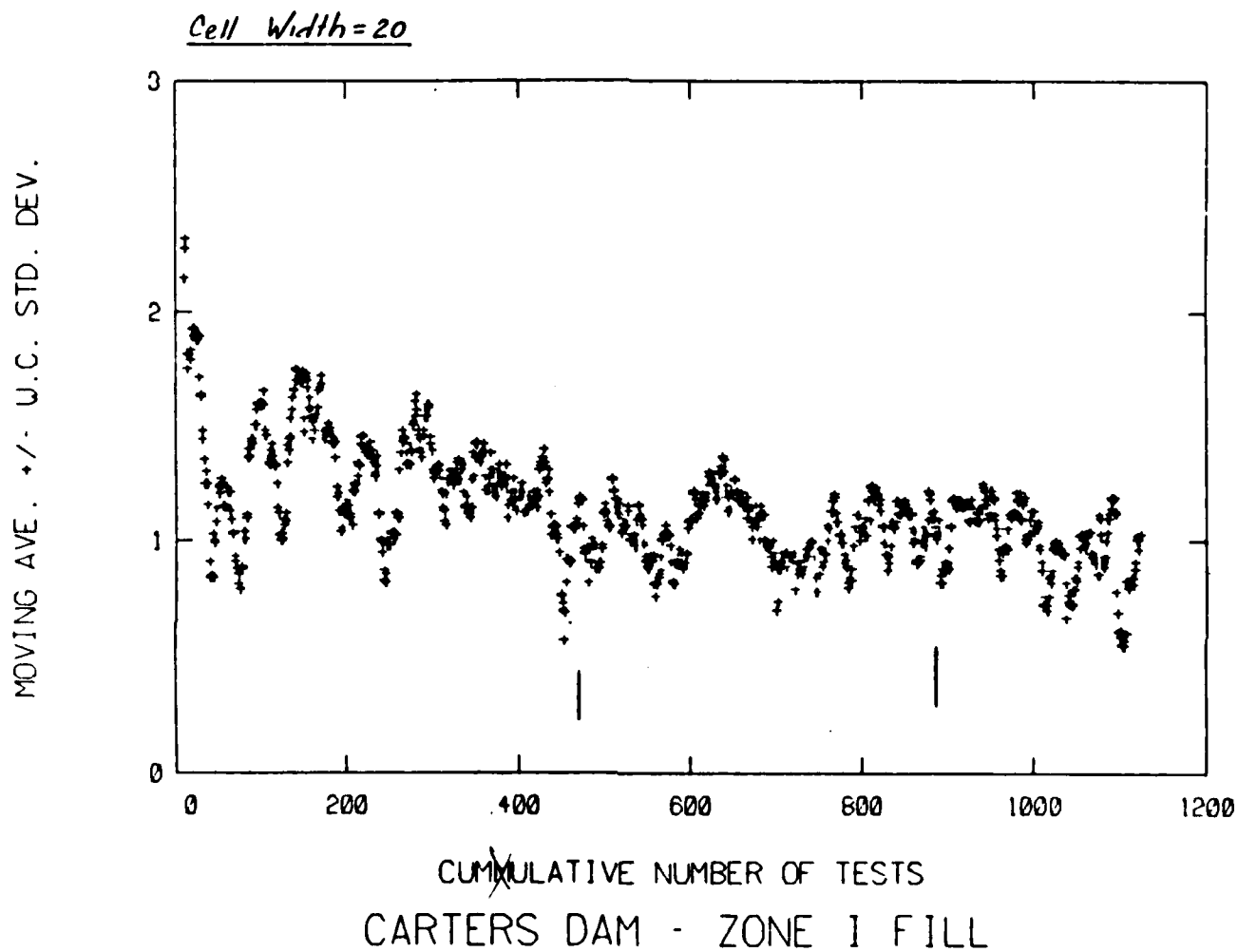


Figure 53. Moving Average Standard Deviation of \pm Percent Water Content Data.

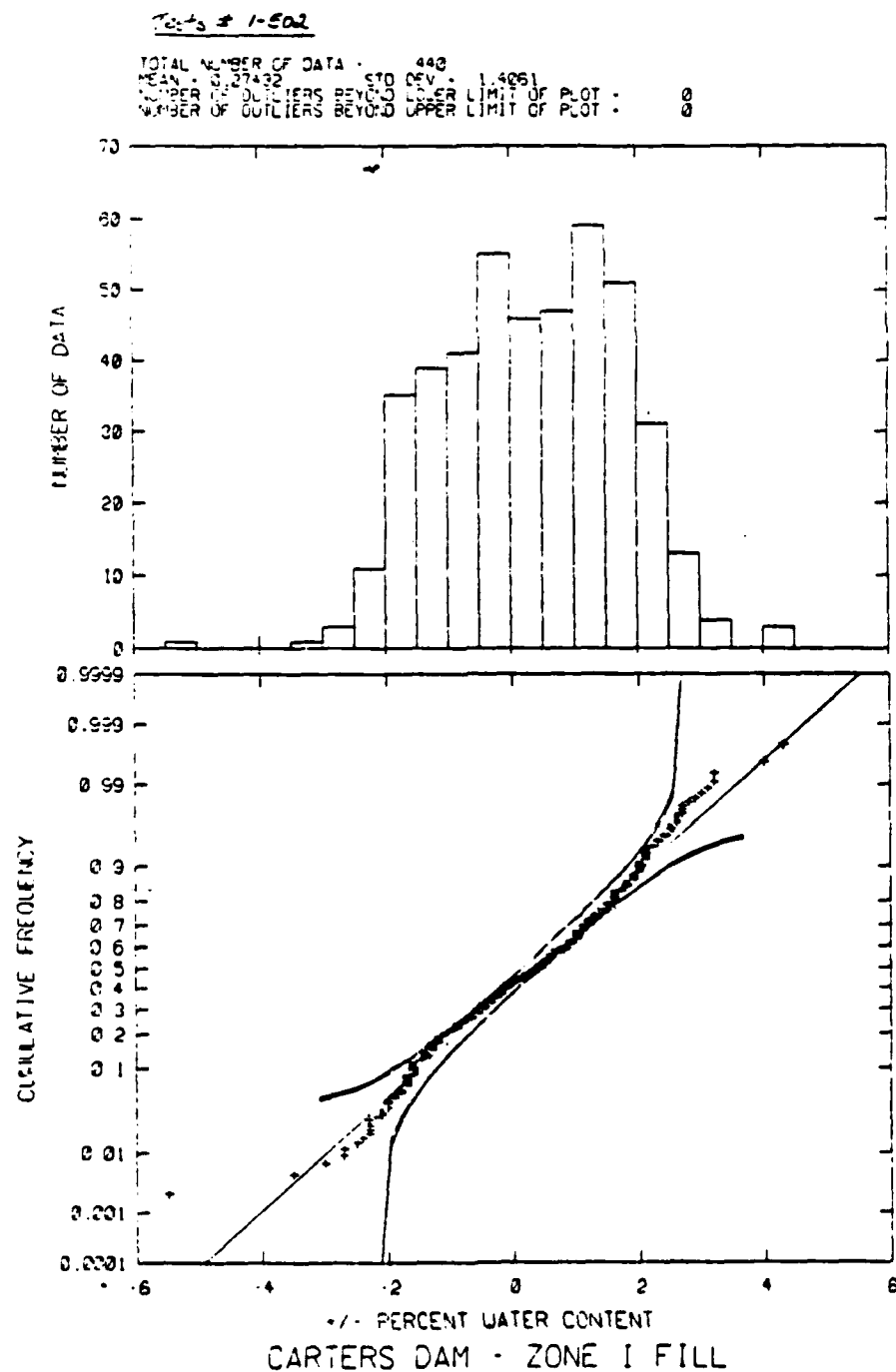


Figure 54. Histogram and Probability Plot for First Period Data.

Task # 900-1244

TOTAL NUMBER OF DATA : 254
 MEAN : 0.61772 STD DEV : 1.0344
 NUMBER OF OUTLIERS BEYOND LOWER LIMIT OF PLOT : 0
 NUMBER OF OUTLIERS BEYOND UPPER LIMIT OF PLOT : 0

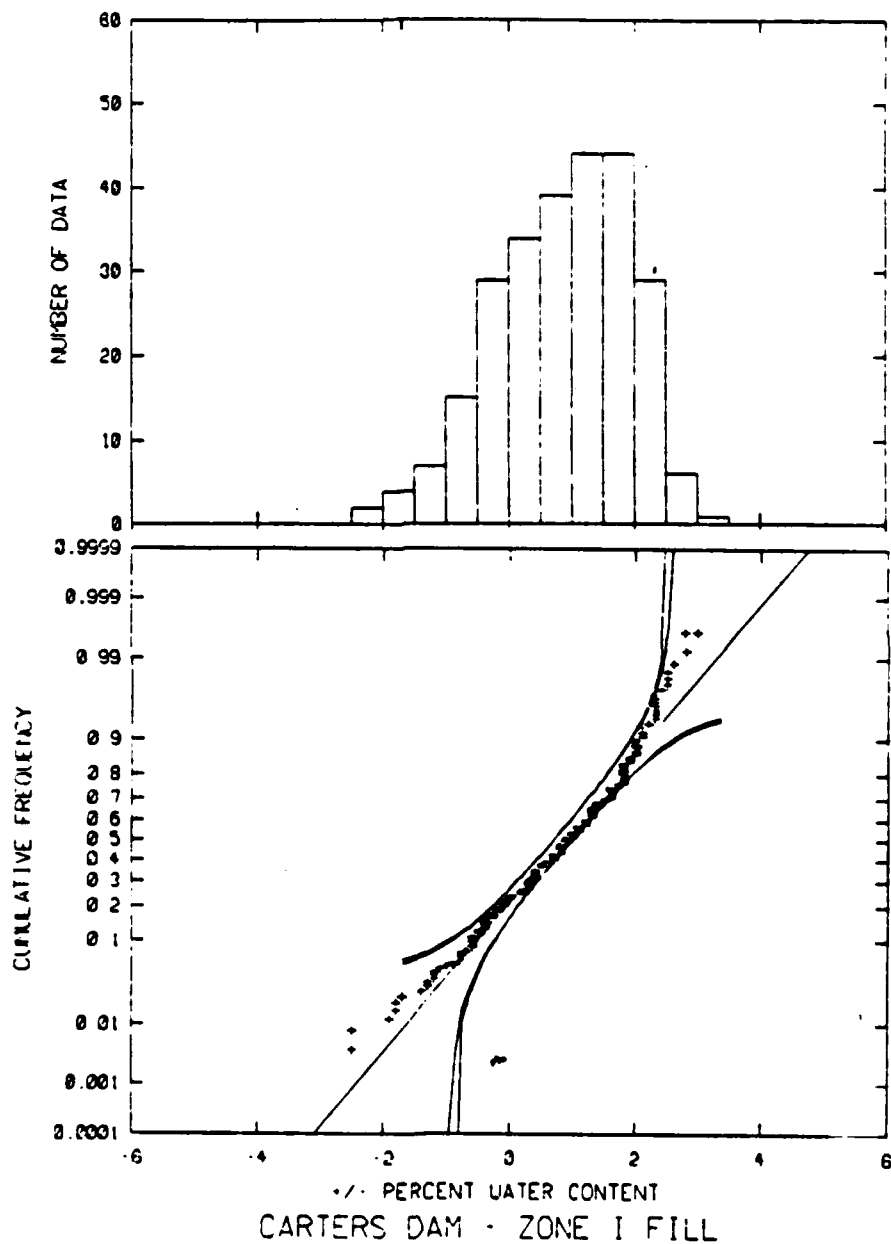


Figure 55. Histogram and Probability Plot for Second Period Data.

Test # 503-985

TOTAL NUMBER OF DATA : 440
 MEAN : 3.38409 STD DEV : 1.8997
 NUMBER OF OUTLIERS BEYOND LOWER LIMIT OF PLOT : 0
 NUMBER OF OUTLIERS BEYOND UPPER LIMIT OF PLOT : 0

☐ = REJECTS.

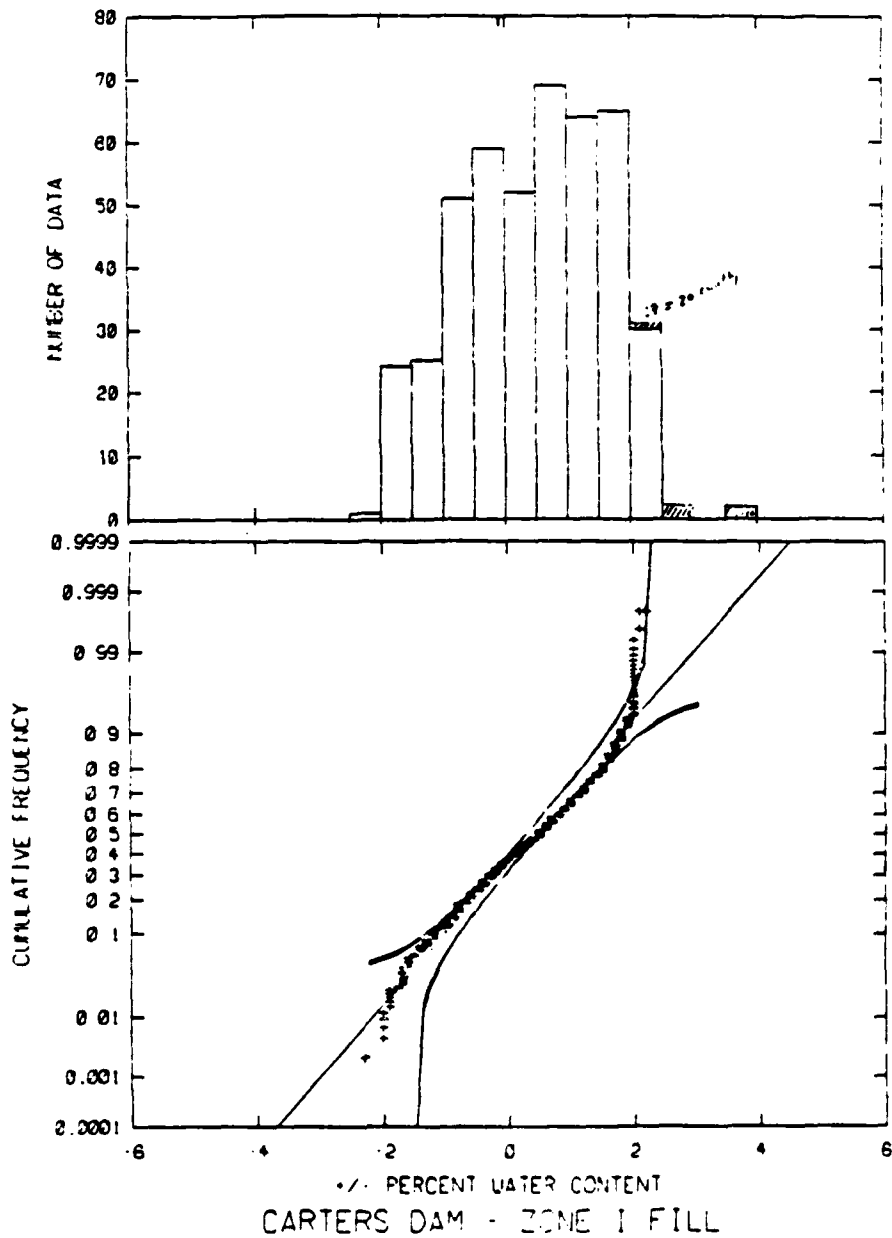


Figure 56. Histogram and Probability Plot for Third Period Data.

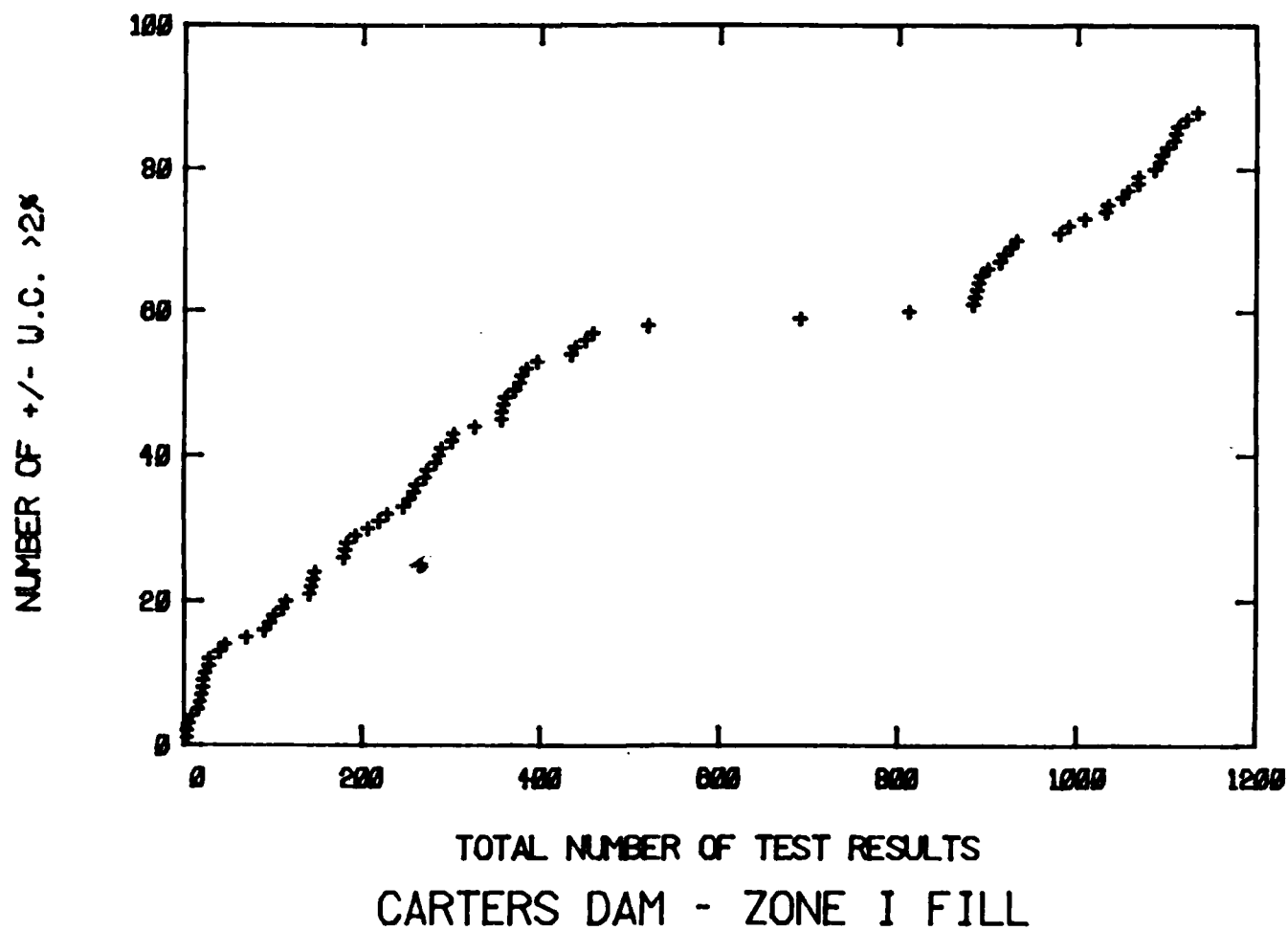


Figure 57. Control Tests Accepted With +/- Percent Water Contents Greater than 2 Percent.

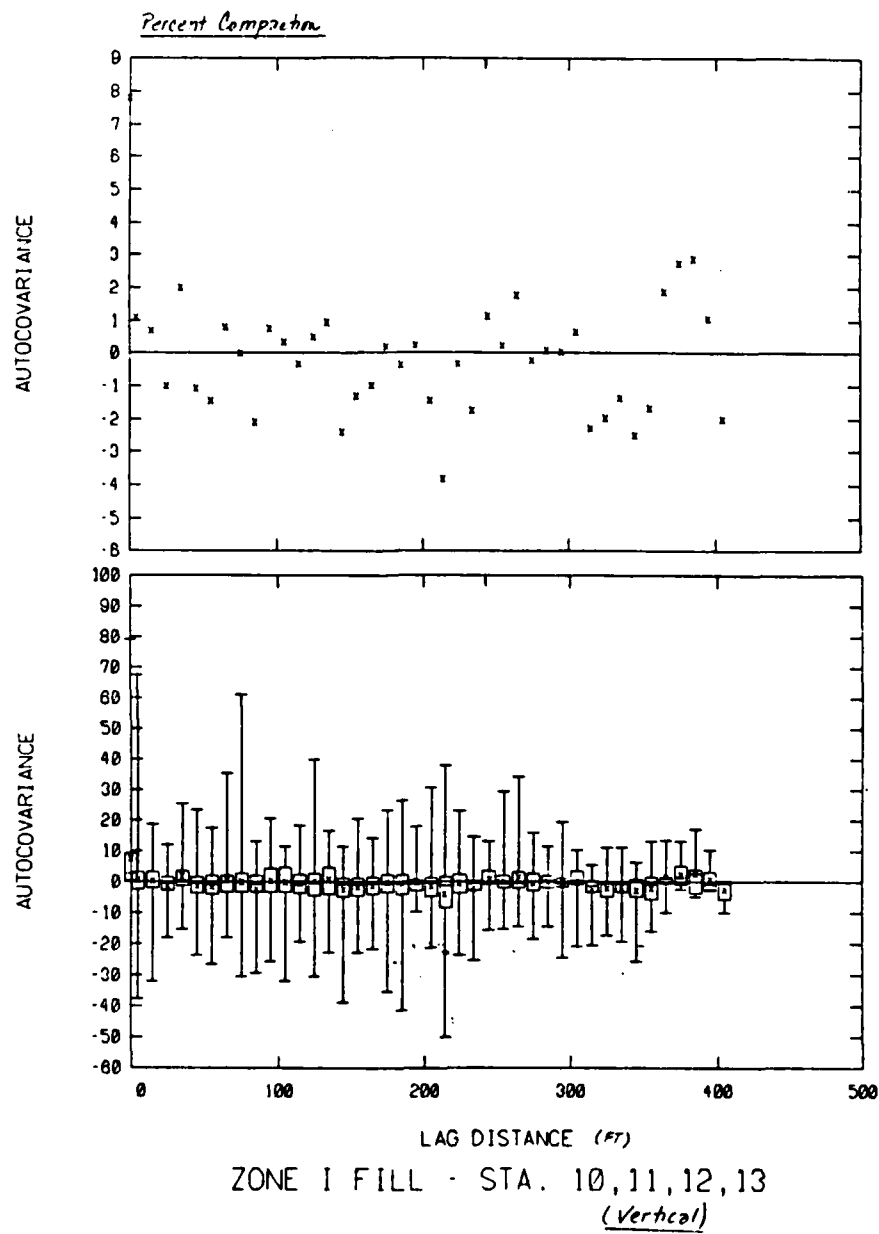


Figure 58. Autocovariance Function of Compaction Control Data as Function of Spatial Separation.

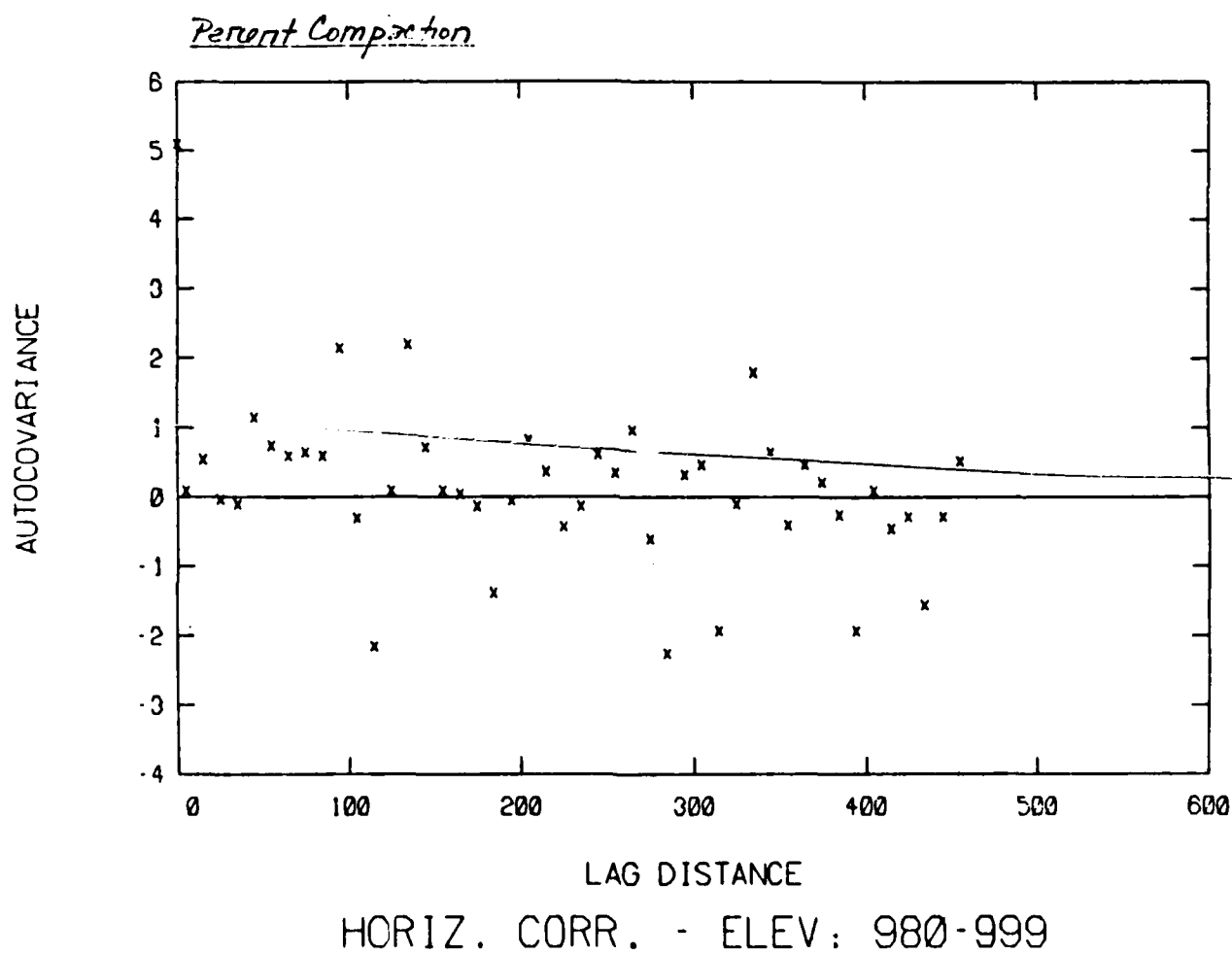


Figure 59. Autocovariance Function of Compaction Control Data as Function of Spatial Separation, Elevation 980-999'.

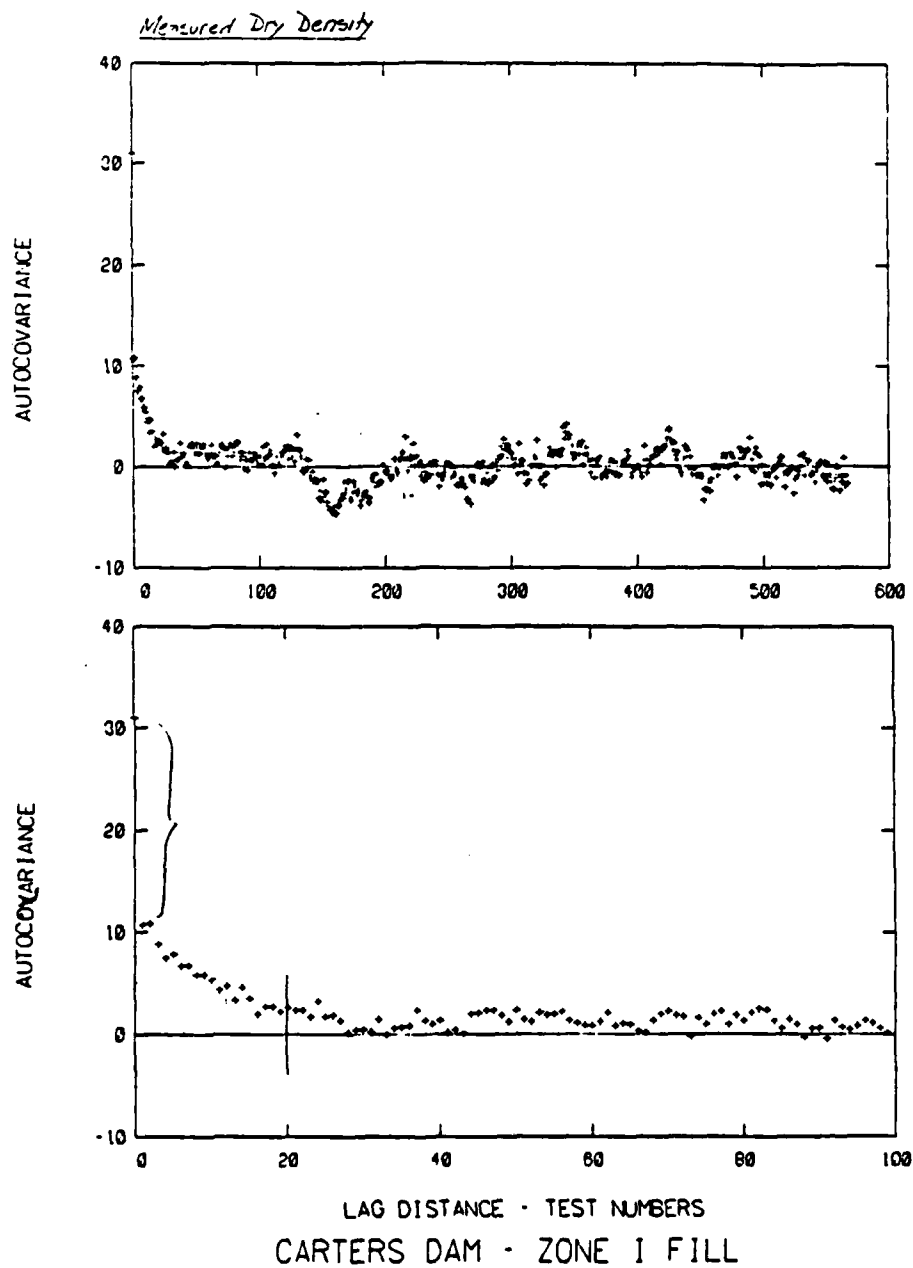


Figure 60. Autocovariance Function of Compaction Control Data as Function of Test Number.

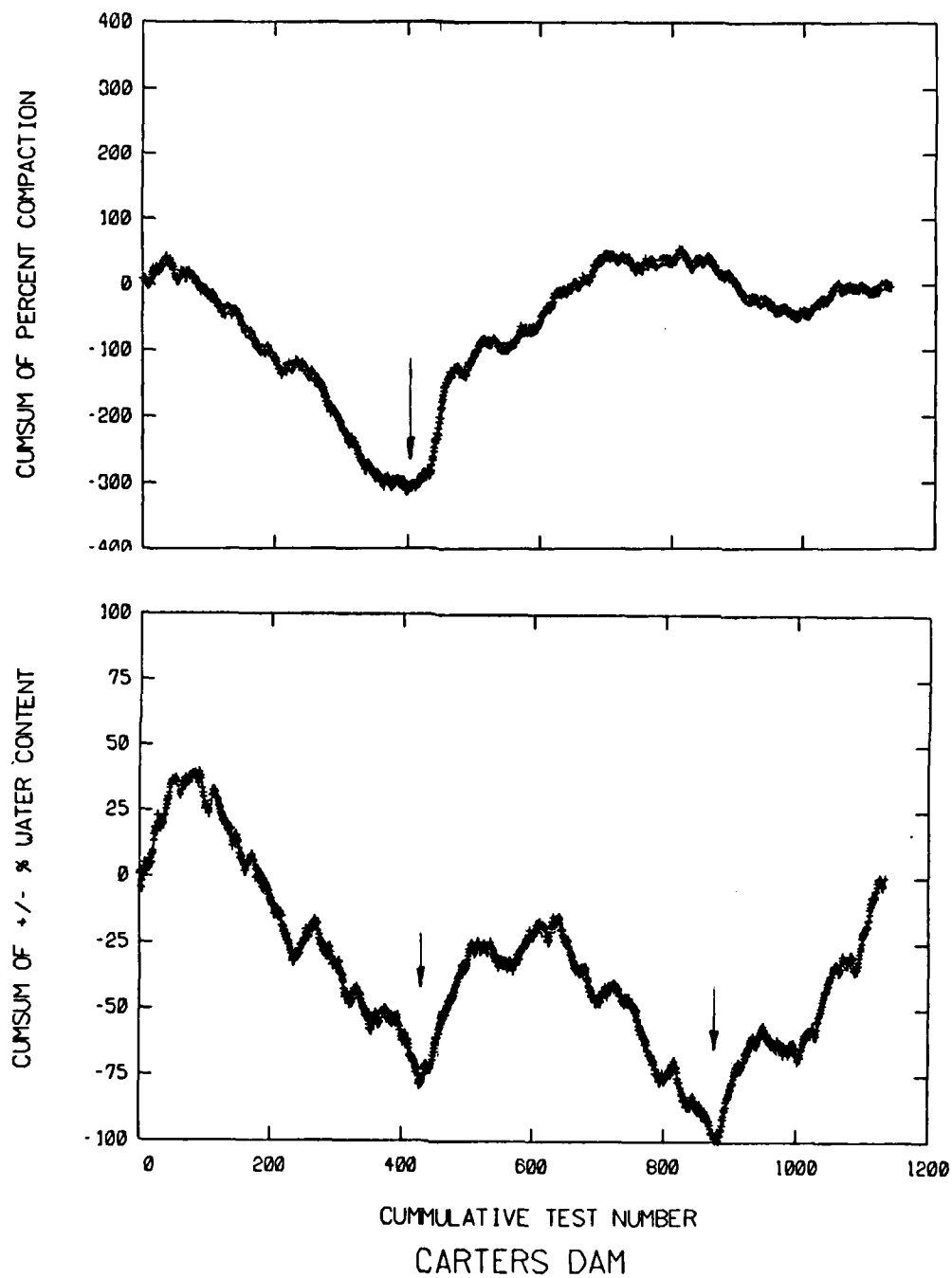


Figure 61. Cumulative Sum (QSUM) Chart of Compaction Control Data.

Log DB85 Analysis on Zone 1 by K-Means; No Weight

Output for each cluster

Cluster#	# of Data	S.S.	Centers
1	16	1.051878	-0.8178277
2	12	0.3948698	0.1424372
3	26	1.605676	0.9616060

Total Sum of Squares	34.49413
Sum of Squares within Clusters	2.952424
Sum of Squares between Clusters	31.54171
S.S.b.C. / T.S.S.	0.9144080

Zone 1 - Core of the Dam -

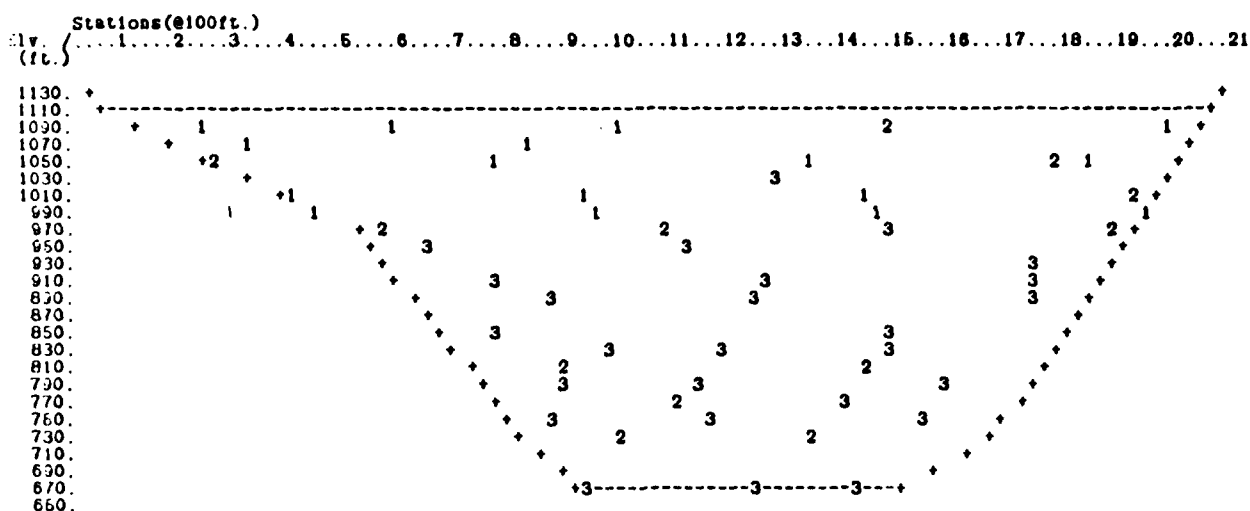


Figure 62. Grain Size Zonation of Impervious Core Based of Statistical Analysis.

PART V -- RETROSPECTIVE ON POTENTIAL INFLUENCE OF STATISTICAL
METHODS ON CARTERS PROJECT

This section considers the influence that recommended statistical procedures might have had on the Carters Project, had they been applied during planning and design phases. Specifically, what decisions about the project might have been made differently, and what benefits might have been obtained that were not?

Peculiarities of the Carters Project
Regarding Statistical Methods

The Carters Project, as any project, is not typical of all USAE dam projects. Carter Dam is the largest and highest structure within the inventory of the South Atlantic Division. Construction of the project progressed without serious incident, and the structure has performed well throughout its operating life.

Foundation Conditions at Main Embankment

First, foundation conditions for the main embankment are excellent. The site of the main embankment was stripped of residual soil and partially weathered rock to found the embankment on hard unweathered metasediments. Core recoveries in the unweathered rock were generally greater than 98%. On the right abutment depth to sound rock varied from 0 to 30 feet (0 to 10m). On the left abutment depth to sound rock varied from 10 to 60 feet (3 to 20m). Few open fractures and no deep jointing were found in the foundation, abutments, or rock excavations. The implications of excellent hard rock foundation and abutment conditions for the main embankment are,

- (a) Little quantitative analysis needed to be or was performed on subsurface data at the main embankment site, and

(b) Little engineering analysis needed to be or was performed on the strength, deformation, or seepage response of the foundation.

As a result, neither mathematical nor statistical methods were important for this aspect of the project.

Seismic Design Second, the Carters site although adjacent to one of the major thrust features of the southeast U.S., the Cartersville Fault, is not in a region of high seismicity. As a result, and given the era of construction and the rockfill shells which behave well seismically, analysis of dynamic response was not critical to design.

Deformation and Seepage Analysis

Third, again given the era of construction, neither deformation analysis nor detailed seepage analysis was performed on the main embankment. It seems probable that Casagrande, as consultant on the project, performed some form of seepage calculations, but these are not in the Corps' records. Thus, the need for geotechnical data to support such modeling and the need for error analysis to interpret the results of that modeling was modest. Today, deformation and seepage analysis are more routine and draw benefit from statistical methodology. The evolution to more comprehensive engineering analysis on all dam projects means a greater usefulness for statistical methods on present projects than on a project of the Carters era.

Embankment Geometry

Fourth, the fundamental assumptions at Carters which dictated later decisions on embankment geometry and construction procedures had little to do with engineering measurements, data, or quantitative analysis. One of these fundamental assumptions, for example, was the frictional resistance that could

be mobilized between compacted soil and rock fill of the embankment and the rock foundation. This assumption was agreed upon by the principal consultant and project engineers based mostly on judgement and analogs with other projects. That judgement turned out to be sound, but quantitative analysis played little role in arriving at it.

Due to such peculiarities of any specific dam project--in the present case of the Carters Project--conclusions of the present study are necessarily biased. Statistics is a quantitative branch of mathematics. Its usefulness lies in analyzing data, calculating the implications of uncertainty, and planning sampling programs. Therefore, on the Carters Project statistical methods are found to be primarily useful in aiding those engineering activities which involved sampling, measurement, and modeling. Statistical methods are found to be not very useful for those activities which do not involve sampling, measurement, or modeling. Since activities which would normally or in present projects involve data and quantitative analysis did not do so in the Carters Project, there was no way for statistical applications to these activities to be tested.

Major Design Decisions

The geotechnical engineering decisions which appear in the records to have exerted the greatest cost influence on the Carters Project were:

1. The decision that the Carters site was superior to the earlier identified Goble site for a large embankment dam.
2. The estimate that insufficient borrow materials would be available at the Carters site to construct the main embankment according to the original design cross-section.

3. The decision to increase the allowable effective base friction angle between foundation and embankment from 30 to 35°, but not to 39.5°.
4. The decision that a core trench was not required.
5. The conclusion that rockfill slopes steeper than 1.8H:1.0V would be unacceptable.

Carters vs. Goble Site

The decision in favor of the Carters site over the Goble site was based heavily on geotechnical considerations. These were that deep weathering at the Goble site reduced the economic viability of constructing a large embankment dam at that location. Simultaneously, preliminary site characterization at the Carters site indicated: (a) rock mass jointing in the foundation and abutments for the proposed dam would require only a "normal grouting program," and (b) sufficient suitable borrow materials from weathered rocks at the site and from adjoining ridges would be available for the dam core, and sufficient rock fill borrow would be available from required rock excavations for the rock fill. The result of these findings was a recommendation of the Carters over the Goble site.

Change of Design Cross-Section

The original design cross-section for the main embankment (1962) is shown as Fig. 63a. In 1965 the cross-section was changed to that of Fig. 63b. The primary reason for the change appears to have been an assessment that available borrow materials at the Carter site, in particular impervious fill materials, would be inadequate for the original cross section. As a result, the second cross section was adopted which was to have a smaller requirement for impervious fill. This decision appears to have been based on quantity

estimates made from boring logs, but also importantly from the decision that weathered rock in the area behind the right abutment might not be suitable fill.

Base Friction Angle

A decision with significant engineering impact on the design of the main embankment was on an allowable value for base friction between the embankment and the rock foundation. Because the stability analyses for the dam were primarily wedge-type analyses, base friction was an important engineering parameter which influenced embankment geometry.

Originally the effective base friction angle was taken to be 30° , in accordance with common design practice. After overburden was stripped from the site and the foundation washed clean, the surface of fresh rock was found to have a saw-tooth shape. Sharp steps along rock joints subparallel to the embankment axis created a rough, angular contact between the rock and the proposed fill.

The assumed internal effective friction angle for the rock fill had been estimated from the test fills to be about 39° . Originally, wedge-type stability analyses on the main embankment had used this 39° also for the base friction. In the design conference of November 1963, Professor A. Casagrande, the principal geotechnical consultant on the project, suggested that this value of design base friction be reduced to 30° . At the design conference of July 1964, after observing the striped foundation, Professor Casagrande recommended that the design value of base friction be increased to 35° . This value was lower than the internal rock fill friction angle, but higher than the common-practice value of 30° used earlier.

Core Trench

The original design for the main embankment considered the use of a core trench. After observing the stripped foundation and reviewing rock drilling records the decision was made to eliminate the core trench as not required, "in accord with present (design) criteria." Rock would be removed back to the base of weathering only. The exact reasoning for this decision at the July 1964 design conference is not detailed in the report of that conference. Presumably, however, the decision was based on the quality of the unweathered rock foundation, the absence of intense or through cutting joints, and the absence of faulting or other major discontinuities.

Rockfill Slope Steepness

The final decision to allow 1.8H:1V slopes in the rockfill was also made at the July 1964 design conference. Unlike the preceeding engineering decision which had major influence on design and cost of the main embankment, the decision on slope angles appears to have been based at least in part on test data and stability calculations. The stated justification for this decision in the report of the July 1964 conference is that the decision was made, "on the basis of judgement of the materials of construction." The relative importance of quantitative analysis and qualitative experience in arriving at the decision is not known.

Influence of Statistical Methods on Major Decisions

The influence of statistical methods on major engineering decisions concerning design of the main embankment on the Carters Project would have been modest or negligible. Of the five decisions noted above, only the second and fifth, were based to any extent on quantitative analysis. The decision in

the second case, on amounts of suitable borrow, appears to have had at least as much to do with what materials were suitable as with calculations of quantities from boring data.

This leaves only the last decision, an acceptable rockfill slope angles, as a candidate for potential influence by statistical method. In fact, the method of error analysis for engineering calculations (Part V; see also, ERROR ANALYSIS FOR GEOTECHNICAL ENGINEERING) may well have provided insight into the confidence with which embankment stability could be predicted. The assessment of uncertainties (i.e., standard deviation) in input friction angles and other parameters for this particular case, however would probably have had to reflect as much subjective content as quantitative data analysis.

Operational Decisions

Comments on how statistical methods would have influenced operational decision and routine engineering activities are divided into the same categories as in Part V: site characterization, engineering analysis, construction inspection, and instrumentation.

Site Characterization and Data Analysis

In order to judge how the site characterization and data analysis program on the Carters Project might have differed had statistical methodology been used, two objectives of site characterization have to be distinguished. One is the reconnaissance function of developing a geometric and geological concept of the site, the other is the testing function of developing engineering properties for analysis. Statistical methodology is not well suited to the first, but is very much appropriate to the second.

At the main embankment site, the purpose of site characterization was primarily the first, reconnaissance. Borings were made to define rock mass geometry and to ascertain whether major fracturing or other discontinuities existed. A major use of the information obtained from the borings was an assessment of the depth of weathering across the site. Few tests for engineering parameters were made. As a result, statistical methods might have provided some insight into marginal improvements of the network geometry of boring locations, which in turn might have provided modest reductions in exploration cost, but it seems unlikely that major changes would have resulted.

At the reregulation dam site the purpose of site characterization was somewhat different, it focused more on engineering properties, stratification, and the definition of solution features in underlying rock. The reregulation dam was a minor part of the overall Carters Project, but its site characterization program is the best example of the type problem in site characterization, testing, and data analysis where statistical methods have a role. For this reason, much of the attention of the site characterization part of the present work was focused on the reregulation dam, even though it was much smaller in scope than the main embankment.

Statistical methodology might have potentially changed the site characterization program at the reregulation dam in at least three ways. First, while statistical methodology would probably not have led to a reduction in the total number of borings at the reregulation dam site, it would have led to a different spatial array of boring locations, which in turn would have provided more information at the same cost. Second, the data resulting from the boring program would have been analyzed somewhat

differently, in that greater attention would have been paid to filtering out noise in the SPT data, and greater numbers of laboratory tests would have been performed. Possibly the greatest benefit, however, would not have come from these rather modest efficiencies, but from increased use of automated data processing which relies on statistical procedures to manipulate data.

The total number of borings at the reregulation dam site seems to have been primarily determined from the desire to have adequate spatial information on the geometry of soil and rock formations underlying the embankment, and not from the need for engineering data to support analyses. From this point of view, the more or less uniform allocation of borings along the proposed axis is an obvious and good plan. However, although more than enough data were collected for statistical analysis, this spatial array is not particularly useful for analyzing the sources of data scatter in the test information taken from borings. Had statistical techniques been used, one or more test sections would probably have been used in which boring locations were nested or otherwise clumped to provide data at a variety of spacings, to perhaps as close as 10 feet (3m). From this information a better estimate of the autocorrelation of soils properties could have been obtained, thus a more precise estimate of the noise contained in the data, and thus a better refinement of the soil profile. It is important to repeat, though, that the total numbers of data collected at the reregulation dam site is more than sufficient for statistical analysis. Statistical techniques would not have demanded more data than that actually collected.

Stability analyses for the reregulation dam were primarily based on laboratory measurements of strength properties for foundation soils and for the compacted fill constituting the embankment. While many tests were

performed, the uncertainties in specific strength parameters (e.g., effective c' , ϕ' parameters for drained conditions) remain high due to limited numbers of tests upon which those specific properties were estimated. This uncertainty in soil properties is offset by the relatively high values of expected factor of safety, and thus would appear to have caused little concern. Better definition of soil properties could have led to less conservative embankment cross sections, were stability considerations the overriding factor controlling design.

During site characterization, a section of the proposed embankment axis was found to be underlain by solution features in the sedimentary bedrock. To define the geometry of the solution features, an intensive program of boring was undertaken between Stations 27+50 and 37+00. This problem is very similar to problems in so-called search theory, in which allocations of effort in space are optimized such that the probability of finding undetected objects is maximized. Limited work during the present project suggests that minor increases in efficiency could have been obtained in locating borings to delineate the zone of solution activity. The use of search theory techniques, however, cannot be categorized as routine.

The most important benefit to have been gained from statistical methods on the Carters Project is also one that is difficult to quantify in retrospect, the increased efficiency of automated data processing. Statistical methods would have allowed many data management and manipulation activities that had been performed by hand to have been automated.

Engineering Analysis

Given the era in which the Carters Project was designed, the main analytical activities of a geotechnical nature on the project pertained to stability calculations for the main embankment and for the reregulation dam. At the time of design, seismic considerations appear to have not been judged sufficiently critical to warrant detailed dynamic analysis. Seepage analysis was not yet routinely performed. Deformation predictions, which were in fact made, were based on "observations and examinations of several rockfill dams" by the principal consultant and senior project staff.

There is no reason to believe that statistical methods would have led to fundamentally different approaches to stability analysis. The models for stability calculations in use in 1964 are not markedly different from those in use today. Today, finite element or related numerical models might be applied to the problem of estimating stresses and deformations in the embankments, but at the time these methods were not common in practice.

What would have been different using statistical procedures is, (a) how the sensitivity of calculated results to uncertainties in geotechnical properties would have been assessed, and (b) how the resulting calculated prediction of performance would have been evaluated and compared with one another.

A very comprehensive set of wedge-type stability analyses was performed on the main embankment. The record of these analyses exists in project files in the form of oversized sheets of graphical calculations. In the Design Memoranda and in published references to the project (e.g. Roberson et al., 1966) only the critical condition (Fig. 37) is typically shown. These calculations would be made on a computer, but even so, the same

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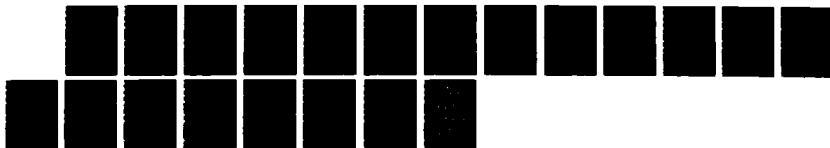
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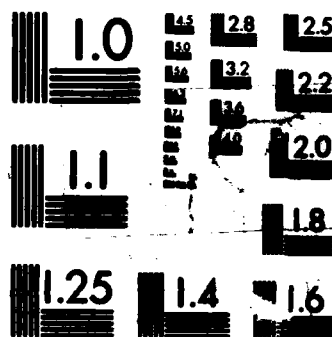
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must be calculated to evaluate the sensitivity of predicted factors of safety to possible variation in soil or rock fill properties. Error analysis would have streamlined the overall effort of assessing the reliability of stability calculations, and would have provided an organized and more traceable record. It is unclear and probably unlikely that the final design for the main embankment would have been different.

One way in which design decisions deriving from statistical analyses might have differed from those actually made on the Carters Project has to do with consistent factors of safety. In retrospect, any conclusion that design decisions would have differed is only speculative, but an example is provided by stability calculation for the reregulation dam. Consistent factors of safety in the sense used here means that performance predictions have consistent reliability.

The uncertainty in strength parameters for the compacted fill materials composing the reregulation dam is smaller than the uncertainty in strength parameters for the subsoils beneath. The coefficients of variation in the former case are in the range 0.1 to 0.3; in the latter case they are in the range 0.4 to 1.0. For a mean (best estimate) factor of safety against strength instability of 2.0, which is not atypical for the reregulation dam, the corresponding reliability indices would be, for failure surfaces predominantly in embankment materials:

$$\beta = \frac{m_F - 1.0}{s_F} = \frac{2.0 - 1.0}{0.3} \text{ to } \frac{2.1 - 1.0}{0.1} \quad (44)$$

$$= 3.3 \text{ to } 10$$

and for failure surfaces predominantly in foundation soils:

$$\beta = \frac{2.0-1.0}{0.3} \text{ to } \frac{2.1-1.0}{0.1} \quad (45)$$
$$= 1.0 \text{ to } 2.5$$

For comparison, consider the one case in which the coefficient of variation for embankment materials was 0.2 and the coefficient of variation for foundation materials was 0.5. To achieve equal reliability, say at $\beta=3$, the best estimate factor of safety for failure surfaces through the embankment would have to be 1.6, while that for failure surface through the foundation would have to be 2.5. Similar consideration of consistent factors of safety have had important influence on design decisions for other dam projects (e.g., SEBJ, 1983).

Construction Control

Were there only one area where statistical methodology would clearly have benefited--or at least influenced--the Carters Project, it would be in construction inspection. As with most projects of the era, and with most projects today, construction quality assurance for engineered fills rests primarily on ad hoc inspection procedures. These procedures are serviceable, especially when the quality of field personnel is high. Nevertheless, they suffer limitations: First, there is no way to explicitly guarantee quality according to quantitative standards; second, results are difficult to interpret after the fact; third, effectiveness is sensitive to the quality and experience of field inspectors; fourth, subtle trends or changes in construction quality may not be discernable in a timely manner.

The last of these limitations is in evidence in the Carters Project compaction control data. The trends in those data are more fully discussed in Part IV. Inspection of the compaction operation used two measured properties to control and also to assure adequate compaction, as-placed dry density and water content. Two specifications were in effect, that percent Standard or Modified Proctor optimum compaction be at least 95%, and that as-placed water content be within $\pm 2\%$ of Proctor optimum. Corrective action was specified to be at USAE expense if dry densities were inadequate but water contents were within specification, and at contractor's expense if water contents were not within specification. The inspection program consisted of purposive sampling of materials as lifts were placed. Purposive sampling means that specimens were selected according to the judgement of the inspector. Purposive sampling contrasts with random sampling in which a prespecified procedure for selecting specimens preclude the inspector from arbitrary selection (see, "Statistical quality control for engineered embankments," (Contract Report GL-87-)).

Figure 49 shows the cumulative number of tests classified by the inspectors as defective, as a function of test number. Test number is an approximate surrogate for time. Defective means either that the specimen had a dry density less than 95% Proctor optimum or a water content outside $\pm 2\%$ of optimum, or both. The declining rate of defectives is a usual occurrence, sometimes described as a learning-curve effect. This has been observed on other dam projects (e.g., Kotzias and Stamatopoulos, 1975).

Figure 57 shows the cumulative number of test results--whether or not classed as defective--with reported water content outside $\pm 2\%$ Proctor optimum. Again, the x-axis is test sequence number (= time). In these data an unusual anomaly is observed. The frequency distributions of data for the

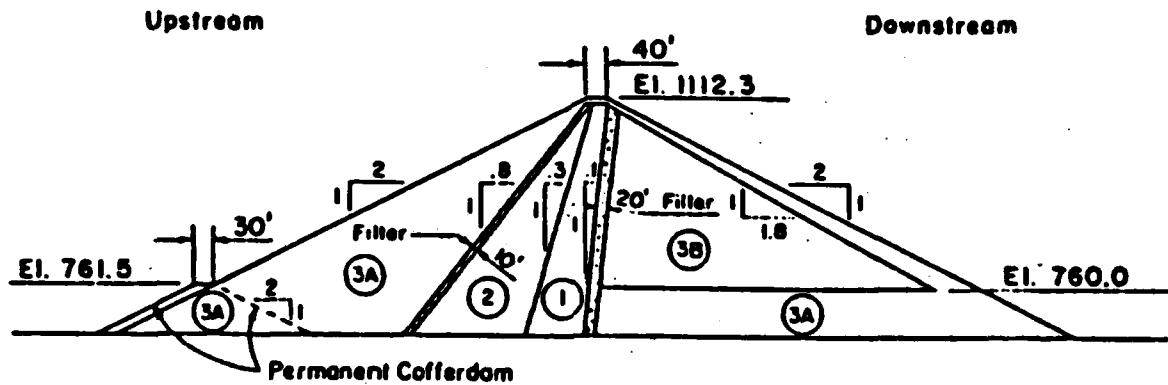
entire project and for increments of the project are approximately normally distributed. These distributions suggest a background rate of water contents outside $\pm 2\%$ Proctor optimum of 10-12%. This rate is observed during the first two years of construction, and again during the last 10 months. However, for a period of 16 months in the middle of the project the rate drops to 0.5%, even though the midrange shape of the frequency distribution of water contents remains essentially unchanged.

Figure 64 shows moving average water content test results averaged over a window of 20 tests. Dates corresponding to period boundaries on Fig. 57 are drawn as vertical lines. In these data the boundary dates correspond to abrupt changes in average test results. Yet, at the same time, the overall average water content throughout each period is the same as in the other periods. The moving average standard deviation decreases progressively as the project continues, but not enough to explain the anomaly.

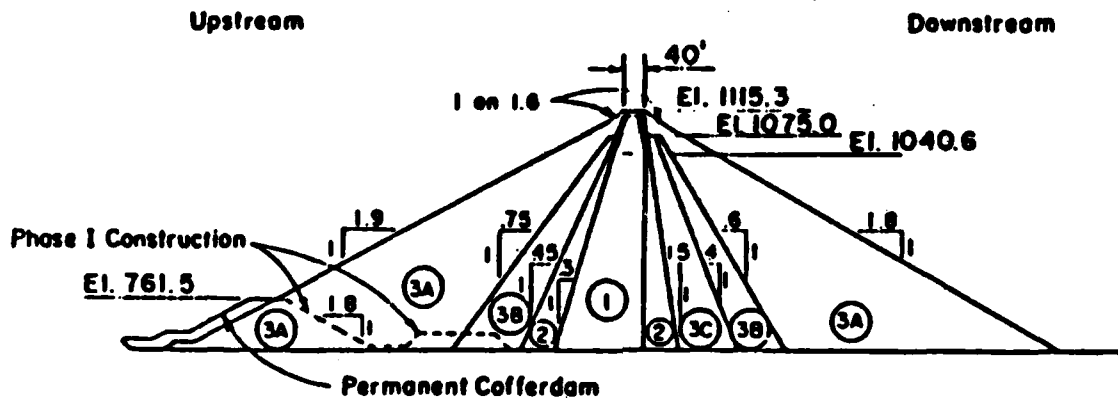
These anomalies could imply several things, some harmless, some not so harmless. In retrospect, the core of the main embankment has performed well in operation, and there is no reason to believe that its construction or engineering properties are inadequate. On the other hand, were such a peculiarity in compaction inspection data to appear during construction on a new project it would present a source of concern. The three obvious sources of such an abrupt change in the statistical properties of the data are, (1) a change in borrow material, (2) a change in construction procedure, equipment, or personnel, or (3) a change in the inspection procedure or way of selecting specimens. The first appears from project records to be not the case. Whether the latter two account for the change could not be determined.

Again, the importance of anomalies in the quality assurance data is not that the embankment is inadequate. The importance of the anomalies is that they were not detected during construction, at least as could be determined from the engineering and construction records. Simple statistical procedures for sample inspection would have provided tools for investigating the variability in QC/QA data as the data were collected, and thereby would have allowed corrective actions to have been adopted if called for. The use of QSUM data plotting, for example, is one technique which would have allowed the anomalies in the data to have been recognized early.

An important change has occurred since the start of construction on the Carters Project. The contractor, not the Corps, is now responsible for quality control of compaction. The Corps exercises a QA responsibility in reviewing contractor data and in inspection sampling. Thus, the need for systematic, efficient, and well defined procedures for real time review of compaction data is more important now than it was in 1965.



A - Preliminary Section (1962)



B - Final Design Section (1965)

NOT TO SCALE

Figure 63. Preliminary and Final Rock Fill Design Sections
(From Robeson and Crisp, 1966)

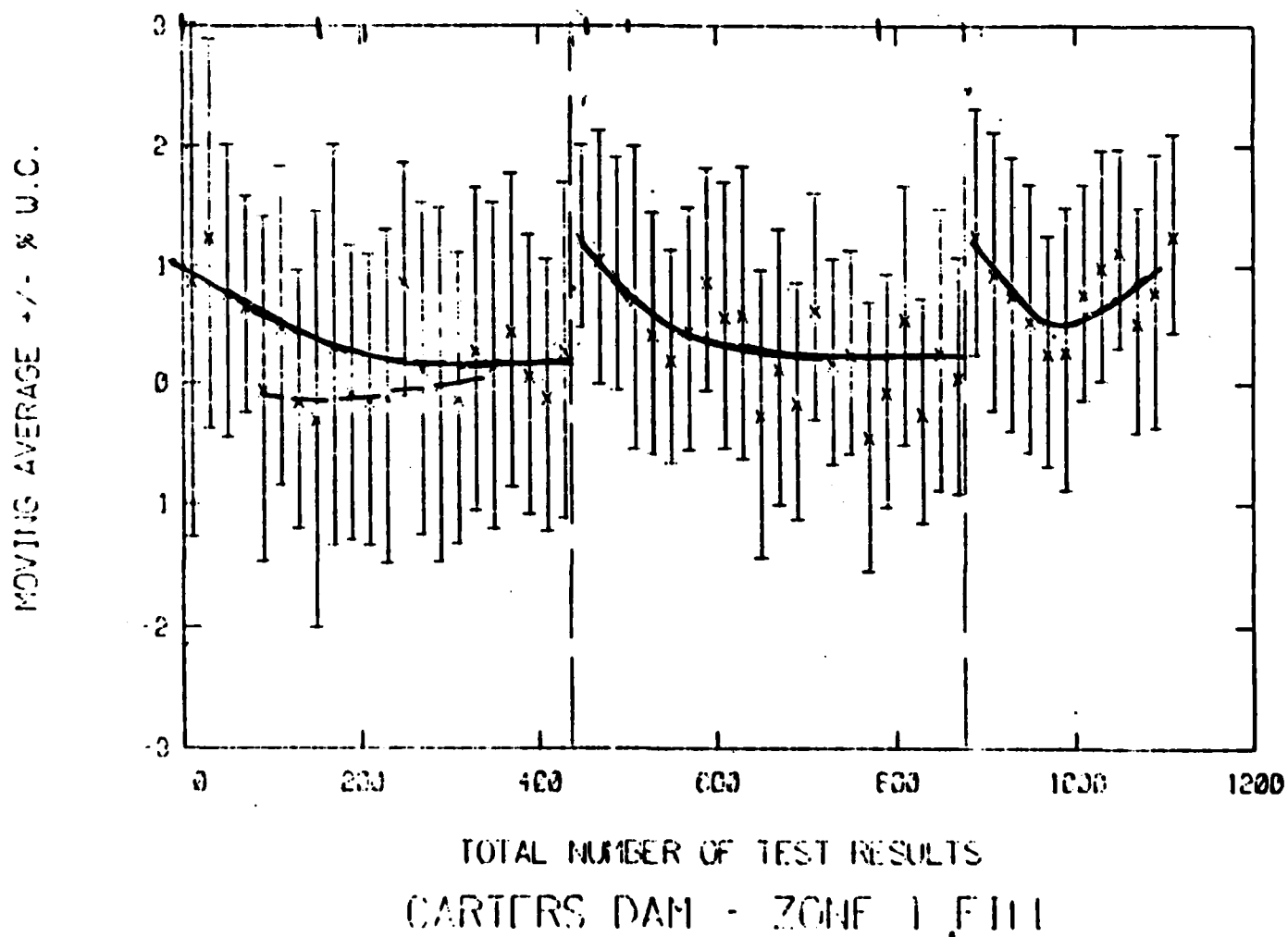


Figure 64. Moving Average Water Contents of Compaction Control Data.

PART VI: IMPLEMENTATION

Part VI briefly considers questions of implementation strategy for introducing statistical methods to geotechnical engineering aspects of new USAE dam projects.

Strategic Decisions on New Technology

As with any organization, the USAE is faced with budget and man-power constraints in developing and implementing new technologies. Statistical methodology is but one of many new technologies that offer potential improvements to current engineering practice. Even within the realm of statistical technology a large number individual methodologies might be considered for adoption. For this reason, research and development decisions necessarily reflect a balancing of benefit against cost.

For those new technologies which offer a favorable balance of benefit to cost, the second decision is how to allocate the effort of development. Who is best able to develop or implement the new technology in a cost effective and expeditious way?

Developmental Stages of New Technology

Any new technology passes through at least three stages in its development. These are shown schematically in Fig. 65. The first is 'inception,' when the idea is new and results come slow and painstakingly. This first stage is characterized by a small number of insightful researchers working typically under small budgets. The ideas are new and conceptual work is needed. The second stage is 'rapid development,' during which advances come quickly. During this second stage the fundamental concept of the new

technology is already developed, and advances are made typically by research groups working under substantial budgets. The third and last stage is 'maturity,' during which most of the main developments have already been made and refinements are now added to the technology. This last stage is again typically worked on by small numbers of people, often highly specialized researchers.

While the schedule of development of every technology differs, at the 'inception' stage useful results might only be expected in the mid-to long-term (e.g., 5 to 10 years). At the 'rapid development' stage useful results are produced and implemented almost continuously. At the 'maturity' stage results may or may not be immediately useful in a cost-effective sense.

From the USAE's view, two things are important about the stages of development of new technology. First, the optimum time for USAE involvement is in the 'rapid development' of stage II. This is the time when the practical return on R&D investment is greatest. Second, the particular strengths of a large R&D organization like the USAE are most fully exploited when critical mass in man-power and budget can be brought to bear on a problem. Again, this occurs during Stage II.

Importance of the R&D Problem

The importance of an R&D problem is judged among other things by, (i) its current importance as reflected in the volume of work associated with that problem area, and (ii) its current growth in importance as reflected by the rate at which work in the area is expanding or by economic or political trends. A convenient way to organize this volume-growth assessment is on a two-way table such as Fig. 66. Various suggestive problem areas are placed on this figure by way of illustration.

An organization like the USAE must evaluate what areas of work, and as a result what areas of technology, are important to its achieving its mission today, and how those areas of importance are changing. By making R&D investments in line with the size and growth areas of critical work the USAE can remain prepared technologically as the civil engineering problems it faces change. The USAE should be targetting areas of work--and the technological development they will require--which lie to the top and right of the volume-growth matrix.

Allocation of R&D Efforts

Decisions on how the USAE acquires technology should depend on an assessment of the relative strengths and efficiencies of the various ways that technology might be obtained. This depends on the technological strengths of the USAE as a research organization, and on the alternative sources of technology. Fig. 67 shows a 2-way plot of USAE expertise or strength along the horizontal, and the benefit of R&D along the vertical. For those areas which lie at the top and right of the plot, the USAE should invest substantially in in-house research. For those areas to the bottom and left, little investment should be made and that which is should be contracted out.

In addition to the question of USAE R&D strengths, there is a second question of the stage of technology development in the outside profession as a whole, compared to that in-house. For those technologies in Stage II of development, the rate of return on investment may be increase as further developments are made. As a result, even if an in-house effort is felt justified, the best first step may be to acquire a certain increment of technology directly from outside and then mount the in-house effort, rather than beginning from a cold start. This is shown in Figure 65. There are

several ways to acquire that increment. One is to hire new staff already conversant with the technology, another is to retrain existing staff either by continuing education or by retaining a consultant, another is to contract for applications of the technology to a USAE project so that in-house learn by doing.

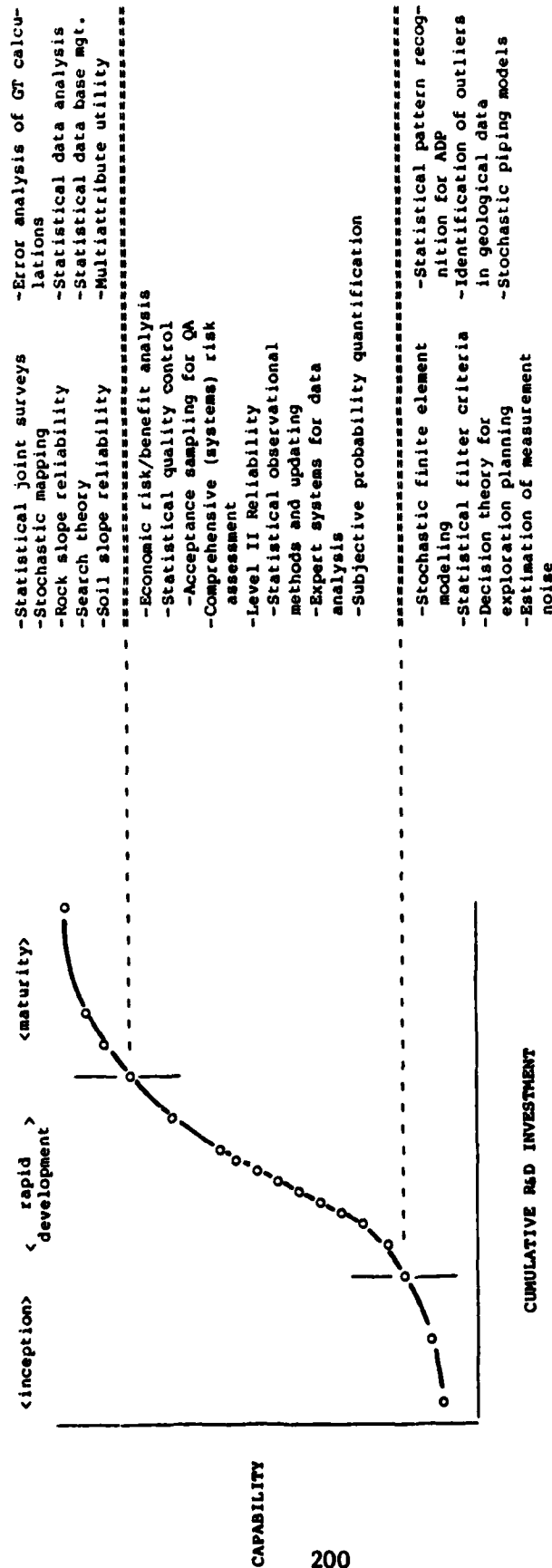


Figure 65 -- Developmental Stages of New Technology

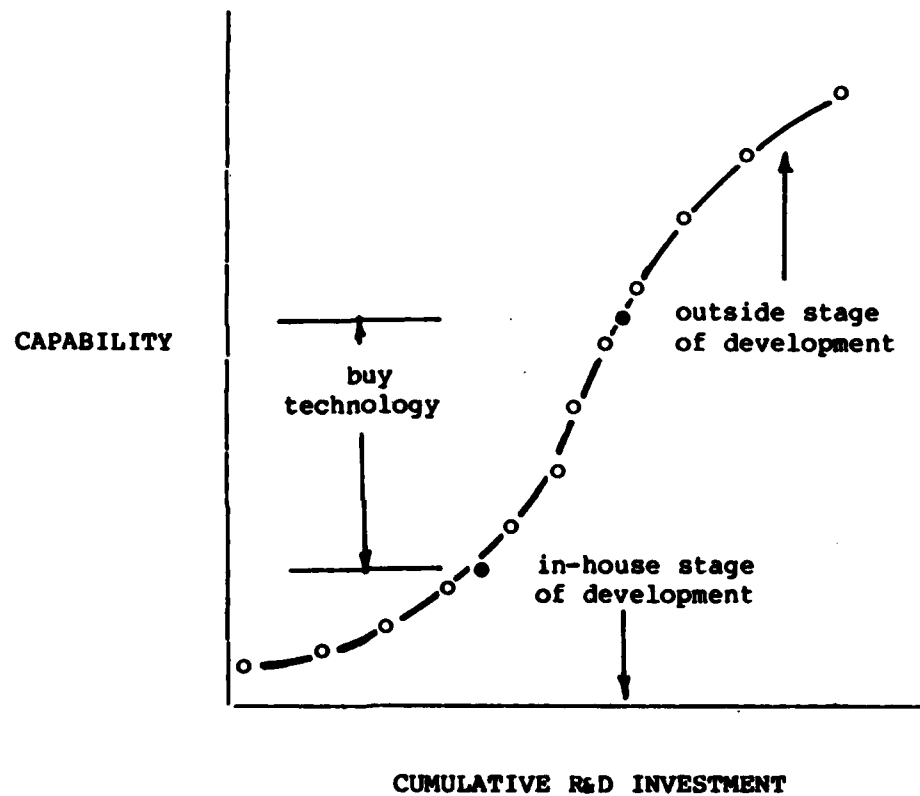
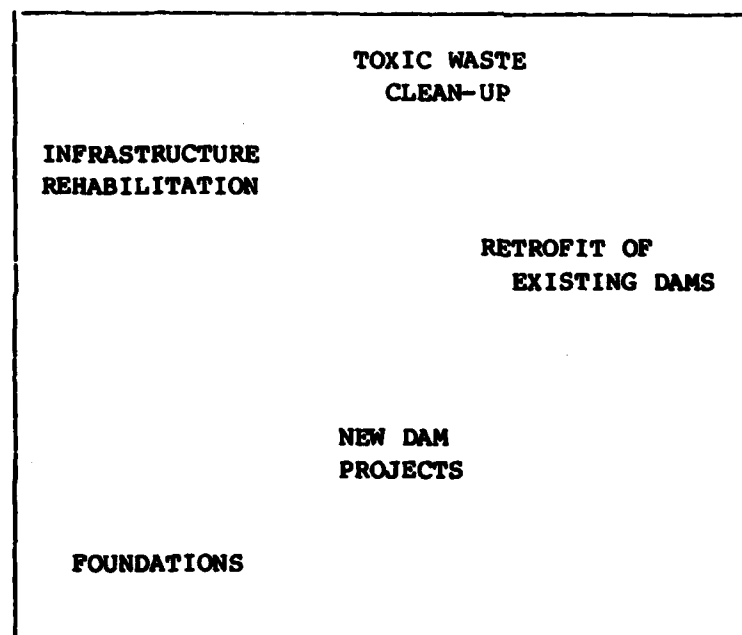


Figure 65 (Continued) -- Developmental Stages of New Technology

Growth
of
Interest



Present Size of Problem
(volume of work)

Figure 66 -- Volume-Growth Diagram for Selected
Civil Engineering Problems

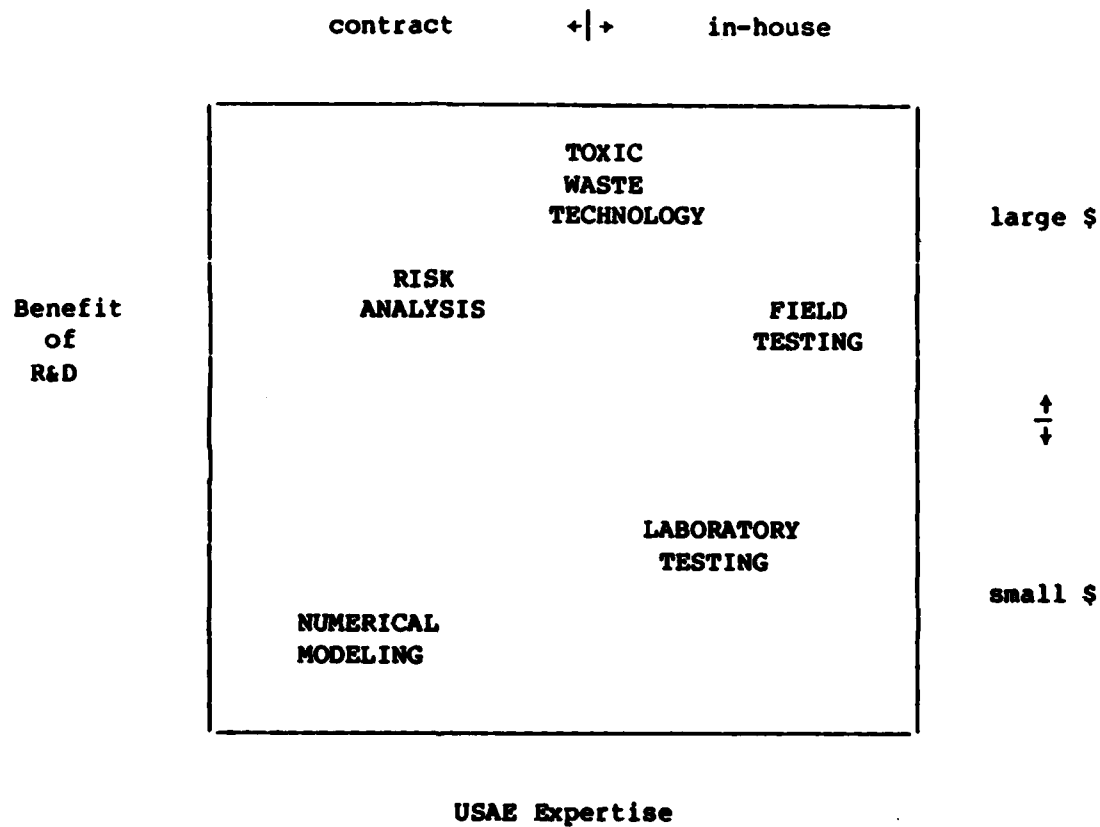


Figure 67. USACE Expertise vs. Benefit of Research and Development Effort

PART VII. CONCLUSIONS

The present project was undertaken with two constraints. First, the study was limited to statistical methods. Second, the study was based on a case history of one USAE project. Certain problems of geotechnical engineering aspects of new dam projects which are amenable to treatment by risk analysis, subjective probability, and other techniques are not amenable to treatment by the statistical techniques considered here. Thus, conclusions of the study pertain to statistical methodologies only. Because the example analyses pertain to only one dam project, the conclusions are biased to some degree by the peculiarities of that specific project.

Review of available statistical methodology for application to geotechnical engineering aspects of new dam projects, and the test application of those methodologies deemed suitable to Carters Project data, leads to the following conclusions:

1. Currently available statistical technology appears to provide benefit in application to routine or operational activities of geotechnical engineering aspects of new dam projects. This benefit is provided primarily by:
 - (a) Increased engineering productivity
 - (b) Facilitation of computerized data processing
 - (c) Enhanced quality control of engineering calculations and construction supervision.
2. The statistical methodologies investigated appear to provide little benefit to major design decisions, as for example site selection or overall embankment geometry. The unsuitability of these statistical

methods for addressing this class of decision is due to the fact that the uncertainties upon which major design decisions rest tend to be qualitative, and tend to involve hypotheses rather than analysis.

3. Present design and specification of quality control and quality assurance programs in construction inspection of compacted earth embankments lags behind practical implementation of scientific sampling in other industries. The QC and QA methods from these other fields are well suited to the needs of geotechnical engineering aspects of new dam projects.
4. Three aspects of geotechnical engineering for new dam projects can be considered now for the implementation of statistical methodology.

These are,

- (1) the management and analysis of site characterization data,
- (2) error or uncertainty analysis of engineering calculations,
and,
- (3) quality control and quality assurance in earthwork.

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